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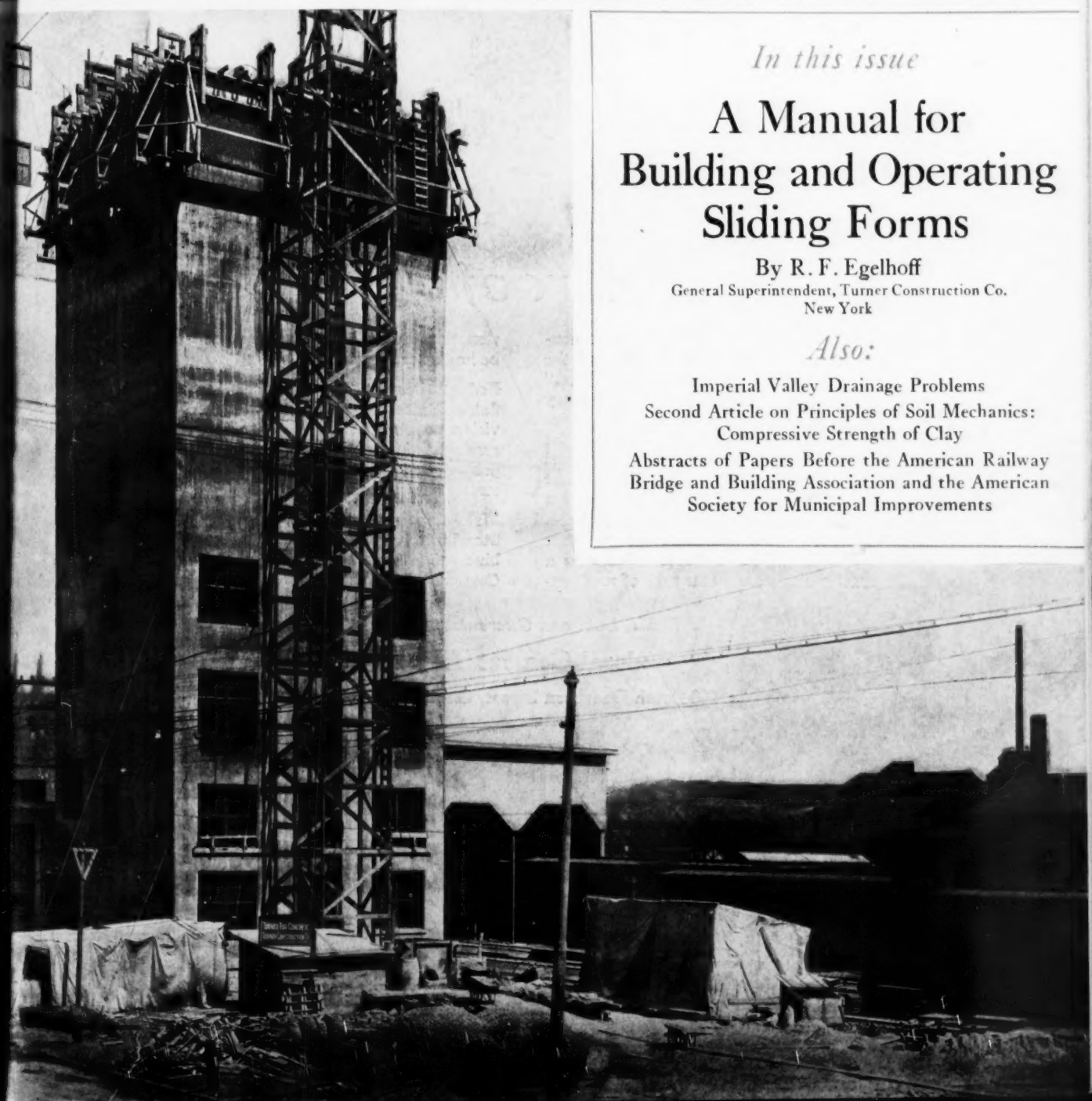


In this issue

A Manual for Building and Operating Sliding Forms

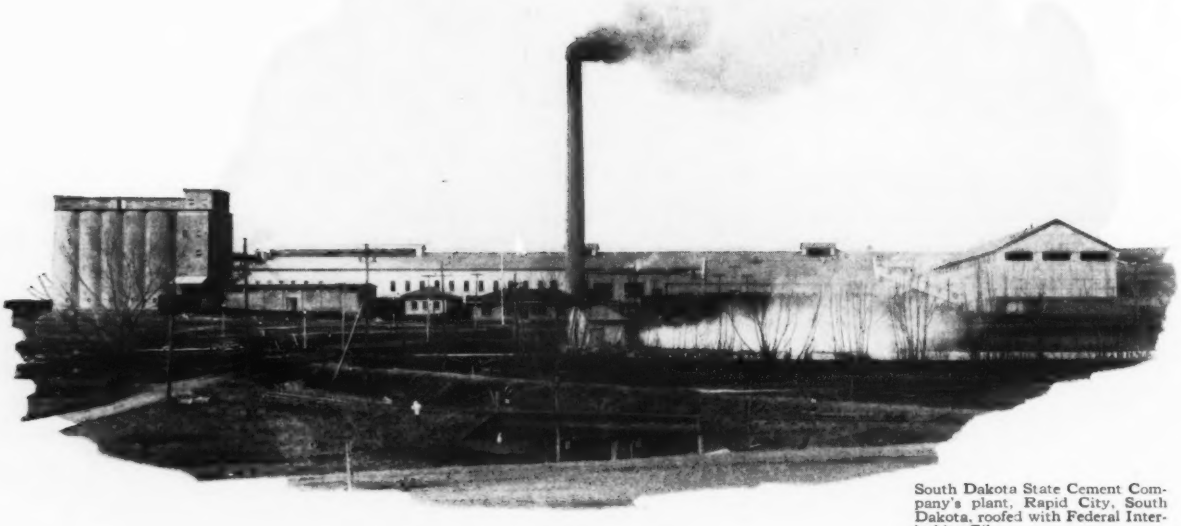
By R. F. Egelhoff
General Superintendent, Turner Construction Co.
New York

Also:

Imperial Valley Drainage Problems

Second Article on Principles of Soil Mechanics: Compressive Strength of Clay

Abstracts of Papers Before the American Railway Bridge and Building Association and the American Society for Municipal Improvements



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ENGINEERING NEWS-RECORD

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DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

FRANK C. WIGHT
Editor

Volume 95 NEW YORK, NOVEMBER 12, 1925 Number 20

Another Contractor Learns a Lesson

NOVEMBER, with its freezing and near freezing temperatures, has begun to take its toll of concrete buildings. The contractor for the Evanston building, whose ruins are shown in the photograph on p. 801, now has ocular, and pecuniary, evidence that concrete does not readily set up in November weather, and that there are precautions in placing concrete these cool days that it is advisable to take. To be sure he might have known this if he had heeded the widely broadcast literature of concrete. His technical journals have been telling him about it for years. His trade and technical associations have hammered it home; the material manufacturers have spent thousands of dollars circularizing him as to the dangers of winter concreting. But he was self sufficient; he knew. Hadn't he been placing concrete for years? What is the use of spending money on tarpaulins and salamanders and tests? Let those theoretical fellows waste their time on such nonsense if they want to; contracting is a practical business. Well, next time he will know better. He has had his lesson. But this burning down a house every time the owner wants some roast pig is pretty expensive business.

Balanced Road Systems

BALANCED road systems are frequently talked of as something desirable which the public cannot be made to adopt. Inferentially this public apathy is conceived as without excuse, the offspring of plain dullness in civic affairs, and we who see clearly congratulate ourselves. Is the question so simple? Recently, taking his own state as an illustration, F. F. Rogers, State Highway Commissioner of Michigan, has spoken briefly on balanced road systems. Considering an area about Grand Rapids it is found that here are 9.16 per cent of the state's population, 8.8 per cent of its valuation, 12.5 per cent of its automobiles and 12.5 per cent of the state roads. The commissioner holds this may very well be called a balanced condition of roads and traffic. Turning to the area about Detroit there is found 38 per cent of the population, 54 per cent of the valuation, 41 per cent of the automobiles and 6 per cent of the state roads. Here certainly there is not balance. Obviously the comparison is not wholly fair but a true situation is indicated which has its counterpart in every state having large cities, where there is no equilibrium between traffic and traffic ways and little reason to expect that there can be. There is no disposition to belittle any effort to relieve traffic congestion even at extravagant expenditure but we have heard so frequently lately the voice of the uplifter crying out the stupidity of the public road official that we are a little irritable. The road official knows the problem altogether better than most others and so is less sure than most others that its solution is easy or even possible.

Six-Wheel Truck Tests

RECENT tests uphold the contention of vehicle manufacturers that six-wheel trucks will solve the problem of heavy freight carriers on the public highway. A six-wheel truck, it appears, tries the road only half as hard as does a four-wheel truck. This is true regardless of wheel spacing except when the axles are set less than 36 in. apart. The nature of the slab stresses is about the same for the six-wheel load as for the common four-wheel load. The tests indicated have been conducted by the Bureau of Public Roads. As most state laws regulating vehicle loads are drawn, the gross load is limited or the axle load, the wheel load and the load per inch width of tire. By adding another set of wheels at the rear where most of the truck load comes, the axles carrying rear load and the number of wheels and the total inches of tire width are doubled. While the six wheels increase the vehicle weight its capacity is increased in much greater proportion. In brief, according to the tests, a heavy freight carrier for public highways is practicable without putting excessive stress on modern pavement types. It remains now for the truck manufacturers to demonstrate the practicability of the six-wheel vehicle in first and upkeep costs, in maneuvering qualities and accident hazard.

For Contractor Responsibility

THE first constructive step toward active co-operation of the public works officer, the contractor and the surety company in improving conditions of contract bidding and award has been taken in California. As the news item on another page reports, these three interests in that state have organized a contract investigation bureau which will attempt to supply the necessary preliminary investigation from which the responsibility of the bidder—and prospective contractor—may be determined. Such information obviously is of the greatest importance to the awarding officer, who wants assurance that his work will be completed and completed satisfactorily as well as cheaply, and to the surety company, which does not want to have to complete work that it has insured. The honest contractor will benefit to the degree that the shysters are kept out of the business by the plan. How much of these benefits will accrue from the scheme remains to be seen but it is evident that it will not work without complete co-operation of all three interests. For that reason its operation will be observed carefully during the trial period. Contractors must be frank in giving information, surety companies must refrain from bonding those contractors who come under the bureau's ban and the awarding officers must, where possible within the law, not be bound to the lowest bidder where he is reported as irresponsible. If this millenial condition can be established—or even approached—other communities will have to come to similar procedure.

The Commission Temporizes

IN REFUSING to take action on the Girand application for a power license at Diamond Creek on the Colorado River, the Federal Power Commission has done a strange thing, and, we think, a bad one. The stated reason is that the commission will continue its policy of taking no action concerning the Colorado River until the states of the basin have agreed on the division of water between them. Neither the reason nor the action can be satisfying to any of the parties concerned, except Arizona; and it does not benefit any of them. Arizona, on the one hand, opposes a project which will yield it no revenue and which may interfere with other projects having the support of the state administration. It is true that the upper basin states also objected to the application, but this was evidently mere rhetorical gesture, as a development in the canyon region can not change the flow of the river above. Apart from the attitude of the several states, the reason alleged by the commission for refusing the Girand license, namely that the Diamond Creek development might prejudice the division of water between the states, is futile, as power use is universally recognized to be junior to irrigation and domestic use, and therefore—aside from the fact that no water is consumed in power development—could not permanently stand in the way of later plans to use the water for irrigation, whether above or below Lees Ferry. If it is the hope of the commission that its action will help to force the Colorado River compact through hostile legislatures, the reason is still less understandable, since the events of the last two years have established quite definitely that the compact is a failure and that the inequity of its terms will make it lastingly objectionable. The commission prejudges the case by stating that the Girand application will retain a preferential status on the list. In thus voicing its intent and yet failing to grant the license, the commission exhibits a surprising lack of courage.

False Doctrine

MANY signs point toward a new animation of the civil engineering mind, and in particular of the structural mind. Not least significant among them is the interest with which the profession has entered into the discussion of working stresses, now current. Two meetings within a week have just given tangible expression to this interest. Such discussion unfailingly leads to progress, through its direct results as well as through its reaction on professional perspective. The rapidity of the progress, however, depends on how sound is the spirit in which the subject is approached. The working stress discussions reveal the existence of surprisingly much backward looking opinion. This is unfortunate, not because the opinion is backward looking, but because it is founded on false doctrine—always a foe to progress. It is to be hoped that the development of structural engineering thought will not be needlessly retarded by its influence.

Not only is the new animation of much meaning to the society in which the profession is united and under whose ægis the two meetings mentioned were held, but it is also of lasting importance from the broad viewpoint of the engineering art as a whole. Working stresses are a contentious subject, and in dealing with it the society has given proof of an aggressive courage frequently absent in the past; this, from the standpoint of the society, is proper matter for gratification. More generally, the current discussion is noteworthy in that it is an integral part of the steady progress of engineering from the earliest days. It concerns increased efficiency of human construction.

All past progress of the builder's art has been pictured in increased efficiency of the use of material. The essential difference between present-day structural skill and that of the old temple-builders of the East lies in this efficiency: in the fact that the builder of old had to use a great deal of material to carry loads which we today are able to carry with very much less supporting substance. Progress in the same direction is still going on, and as year by year the powers of structural engineering become greater this progress inevitably seeks for expression from time to time. It is doing so now in the discussion of working stresses, an effort to crystallize into a form available for direct human service the past quarter century's advance in engineering and constructive abilities.

There is natural disagreement about the raising of working stresses, much of it arising from individual leanings and different estimates of the many intricate factors involved. Such disagreement is commendable, and leads to more careful study of the component questions. Certain of the negative views, however, rest on the assertion that there is—and will always be—some unskilled or incompetent engineering, and that on this account working stresses should not be increased. Those who argue thus do not contend that there is more incompetent engineering now than formerly, nor that old-established working stresses are safe for use with incompetent engineering; at most, they claim that the reserve margin given by the old working stresses may be sufficient to save the day in an occasional instance of this incompetent engineering, and thus give comfort to the fearful and timid. They are silent about the waste to which competent engineering and the world at large is subjected by too low working stresses.

In one respect, we have here nothing other than the inconclusive plea of conservatism, Let well enough alone. But more critically regarded the doctrine is false, because inconsistent in its own facts and claims, and because it belies all the experience and facts of constructional history throughout time. It is destructive because on its basis there can never arise an advancing capable art. It is more than conservatism, it is technical Toryism, a spirit hopelessly incompatible with progress.

It is false to charge that engineering is incompetent in whole or in part, and that low working stresses and large safety margins are necessary in order that the alleged incompetent engineers may serve the world as well as the competent. The incompetents to whom the negative argument refers are not engineers, and the argument really refers to quackery and not to low degree of competence. Thus the engineers who present the argument are working in support of quackery in engineering, and are consciously providing for its continuance. When pressed they may concede that quackery cannot be saved from disastrous results by low working stresses, yet they adhere to their argument.

Moreover, the essential requirements of good engineering efficiency are opposed to this defence of incompetence. The resources of the modern art concededly

are far ahead of those of thirty years back. An art, however, that does not base its practices on adequately full utilization of its powers can not long flourish. The argument against working stresses as high as good engineering permits is therefore a challenge to progress.

But it is a fact also that in every stage of construction the world has always utilized its material at the highest efficiency which the then current state of the art permitted. Builders in all ages constructed as great structures, as bold structures, as economical structures, as they knew. Under modern conditions, it is suggested, such devices as building laws serve to enforce a greater conservatism. Is this true; and is it possible? The widespread disregard and violation of building laws seems to be evidence to the contrary, and seems to show that the demand for utilizing the full powers of the building art is dominant, and that ingenuity finds ample means—even within the bounds of reputable engineering—to nullify unreasonable codes.

There is no question as to the ultimate advance of structural engineering and as to the continued dominance of the principle of full-efficiency construction. The advance for some time to come, however, will be contingent on whether sound or false doctrine proves of greatest immediate influence. Sound doctrine will lead to improved practice, as well as to progressive thought and research. If false doctrine controls, the result will be stagnation.

Electrically Welded Pipe

A BOARD of well known consulting engineers has approved the awarding of contracts for 85 miles of electrically welded pipe on the Mokelumne project in California which is to provide a water supply for the East Bay municipal utility district. The pipe diameters are to be 60 to 65 in. and the unit stresses under service conditions will range up to 14,000 lb. per square inch. Other features of the scheme are reported in *Engineering News-Record*, Oct. 8, p. 608. Coming as it does after the several cases of failure that have occurred in California hydro-electric penstocks in recent years in hammer-forged welds, this decision to venture into the field of electrically welded pipe for heads ranging well up toward 400 ft. assumes considerable importance. Long supply pipes differ materially in requirements from penstock lines and yet there is enough in common to make progress in the one field of great interest in the other.

The Mokelumne contracts are so worded that if the pipe sections do not pass the specified shop tests the contractor may be required to substitute riveted pipe, for which he will be allowed an additional 40 per cent on the contract price. No tests that would indicate the relative life of welds and plates have been specified. Indeed, there may be no tests that would give an indication of the life of the welds in this sense. Electrical welds in thin metal pipes—say up to ¼-in. plate—have been used extensively. There is much more confidence in the welds in these lines than in electrical welds in thicker metal. Only limited experience is available for the latter, at least in this country. Experience with the thin pipes, however, has been chiefly in water-supply lines, where the metal, for mechanical reasons, is much thicker than that required for withstanding the pressure alone. Welds in such lines are not subjected to the severe service conditions of penstocks.

Despite the fact that good welds may show a tensile strength exceeding that of the adjoining plates there is serious question as to the bending resistance of electrical welds of the kind with which we have thus far had experience. One of the difficulties in welding of this kind, particularly in the thicker plates, is that the high heat required for quick fusion results in a change in the metal itself which makes it structurally different from the adjoining wrought metal of the plates. Recently several manufacturers have announced successful electric welding with thick plates, claiming that new processes make it possible for these welds to pass bending and other tests that welds made by other processes could not meet. This is encouraging news and may mean much to penstock builders if and when there is general confidence in the durable quality of such welds.

The $4,000,000 saving that would attend the use of electrically welded pipe instead of riveted pipe on the Mokelumne is a powerful argument, and it is to be remembered that this is a supply line and not a penstock. Just what assurance has been given as to the durability of these welds is not yet known, but so far as other parts of the country are concerned experiments or tests that would give assurance as to the relative life of the welds would greatly strengthen confidence in this type of construction.

Sliding Forms

KNOWLEDGE of many operating details is necessary to successfully work with sliding forms. The form is an operating mechanism and not primarily a static structure as is the fixed form. Its structure then partakes in some degree of the characteristics of machine structure in flexibility. These requirements are excellently brought out in the article in this issue on sliding form construction and operation. Work with sliding forms however is primarily an operating and not a design task and the great value of the article is that it gives more operating data than perhaps anything previously printed about sliding forms.

Primarily the article makes plain that there must be precision of operation and this is not a light requirement to attain. Insignificant in their aggregate of timber as the illustrations on another page show sliding forms to be as compared with fixed forms for the same structures, they have, contemplated as traveling units, a large degree of unwieldliness. No chances can be taken by the solidifying concrete of shock or crowding by the moving form. The molded wall is far from full strength as the form sets it free. There must be precision of form operation.

A less expected lesson of the article is that the nature of the concrete mixture itself is a vital factor in the success of sliding forms. The kinds of aggregates, the proportions and the consistency of the concrete have to be adapted to sliding form work to attain the highest efficiency. The article tells what kind of concrete works best. It tells how to obtain precision in form operation. Its chief value is in these two accomplishments. There is however wide hopefulness indicated that the practical usefulness of the sliding form reaches out beyond the molding of circular windowless bins and stacks to building construction where the interior framing is reasonably regular and the walls are not too broken in surface. This is the goal of economic perfection in sliding form construction.

A Manual for Building and Operating Sliding Forms

Fabrication—Placing—Filling—Jacking—Swelling—Leveling—Bending and Placing Steel—Crew Organization—Plant—Aggregates—Mix—Practice in Structures With Columns and Floors

By R. F. Egelhoff
General Superintendent, Turner Construction Co., New York, N. Y.

SLIDING FORMS differ from stationary forms in the respect that they are not erected, filled, and removed after each fill, but after being erected for the first fill the form is lifted vertically by jacks and the filling is a continuous operation until the structure has been concreted to its highest level. By making the lower section of the form slightly wider than the upper section, the form as it is raised vertically will lift clear of the concrete wall surface. Fig. 1 shows a section through an exterior wall form; the form is ⅜ in. wider at the bottom than at the top.

The type of jack to be used for lifting the forms is largely a matter of individual preference. Fig. 1 shows a common type of screw jack. There are a number of pump jacks on the market for which very satisfactory service is claimed. The screw jack produces a slow steady movement, being threaded to lift ¼ in. for each complete turn. The pump jack lifts ¼ in. for every stroke. It does not have quite as steady a motion as the screw jack, but it requires less labor.

The illustrations referred to in the description of sliding forms for circular bins are of cement bins for the Whitehall Cement Co. at Cementon, Pa. These bins consisted of 9 cylinders 32 ft. in diameter and 60 ft. high. They rested on concrete foundations which were brought up to one level for three bins and to a level 2 ft. higher for the remaining six bins. Fig. 2 shows the sliding forms partially placed for the first fill, and Fig. 3 shows them 40 ft. above the foundations.

Making the Forms—Fig. 1 shows the jack clamps or yokes. These clamps are to prevent the wall forms from spreading, and to lift the forms. The upright posts of the clamps carry practically no load vertically; the entire load is carried by the ¾-in. lifting rods which are bolted through the head pieces of the jacks. Rigidity in the clamp is essential. In case the clamp becomes wobbly by much use, bracing is added to hold it rigid.

Fig. 4 shows the inside and the outside form. Note that the bottom ring wale of the inside forms is made slightly smaller than the upper ring wale so that the form may lift free from the concrete as the concrete gets its set and the form is jacked up. The form, ordinarily 4 ft. in height, should be so built that the width of the forms at the bottom will not be more than ½ in. greater than the width at the top when erected. As the 1x6-in. or 1x3-in. staves which form the face of the form will swell by absorbing moisture from the wet concrete, provision has to be made for this swelling. Fig. 4 shows how it is done.

Fig. 5*A* shows the usual manner in which circular bins are connected to one another. Due to continued swelling of the staves of the exterior circular form, it will be found that as the forms are jacked up the condition at this point will gradually become as shown in Fig. 5*C*, the circular staves having forced the staves in the connecting form in at points *a* and *b*. To eliminate this possibility a split stave should be used at points *c* and *d*. This stave may be split as shown in Fig. 5*B* with piece *f* nailed to the ring wales and piece *g* held in place only by cleats to the adjacent staves. Piece *g* should extend 1 ft. above the top of the other staves. As the circular staves swell and tend to force the form out of shape at points *a* and *b*, the cleats can be loosened and stave *g* can be driven down, thereby relieving the pressure on the connecting form at *a* and *b*.

The ring wales are first made, care being taken to

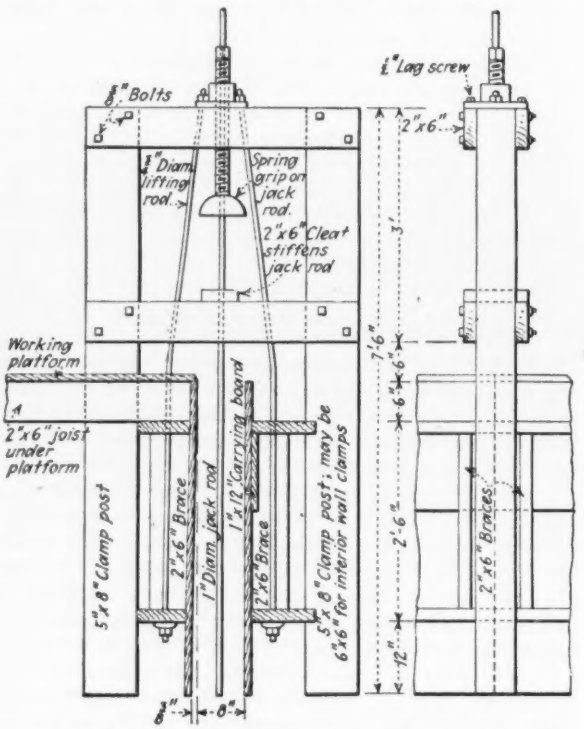


FIG. 1—SECTION THROUGH WALL FORM WITH JACK CLAMP IN POSITION

make them absolutely square, or circular, as the case may be. They are then supported at the desired height on a perfectly level platform and the 1x6- or 1x3-in. staves are nailed to them using either 8d. or 10d. nails. When wales can not be made of single-length pieces, the lengths must be so chosen that the joints will center on jack clamps when erected. A certain amount of flexibility in the forms is produced by having the joints in the wales come directly under a jack clamp. A rigid form will rack and bind when it is jacked.

In circular forms the width of the wale pieces is dependent upon the diameter of the bin and on the spacing of the jack clamps. Segments cut out of the outside wale pieces to form the circle may be strapped or battened to the back of the wale pieces from which they are cut to add strength. In forms for bins of large diameter, segments cut from the outside wale pieces may be strapped or battened to the inside wale pieces to form the inside circle.

After the forms are made the contact areas are given two coats of crude paraffin oil to prevent them from absorbing moisture and to prevent the concrete from sticking to them.

Placing Forms—Sliding forms are always erected either on some suitable foundation slab or on a floor slab from which they are to start their travel upward. As it is absolutely essential that the forms start out from a truly level position it will save considerable labor in the erection to have the slab from which the forms start as nearly level as possible. First the inside forms shown in Fig. 4 are set in position and

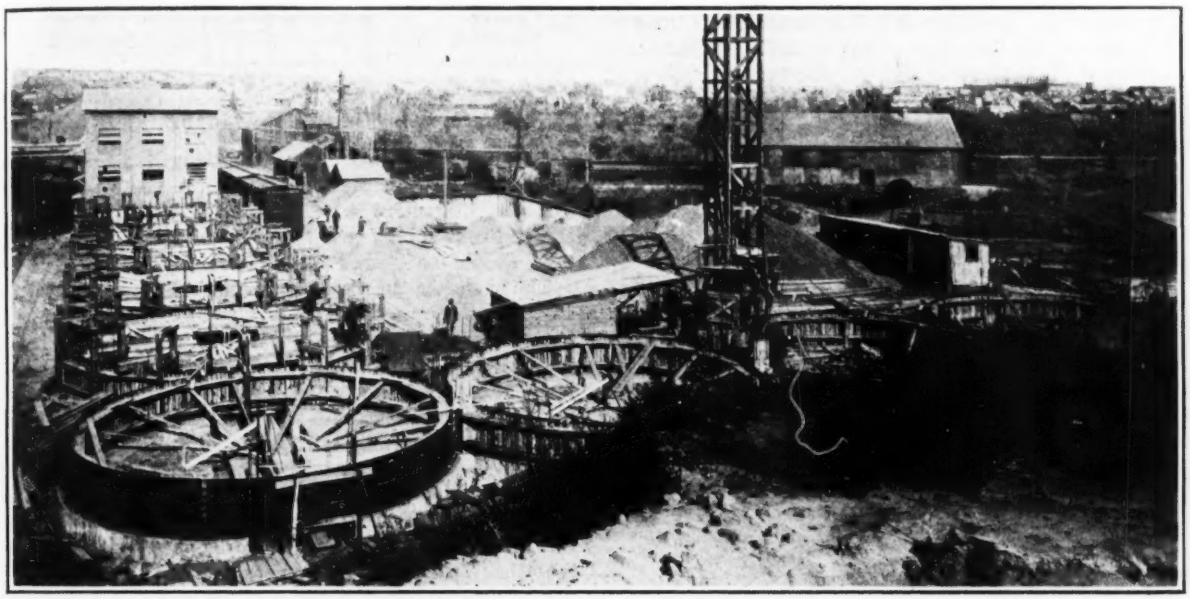


FIG. 2—STARTING MOVING FORMS ON NINE CYLINDRICAL BINS

accurately located by means of lines drawn on the slab. They are then held in position either by bracing to the slab or by bracing from one form to the other. Before this bracing is done, however, the forms must be shimmed up with wedges so that they set level. The outside forms are then put into position and leveled and plumbed. These forms must be set with the face plumb, as all of the clearance is taken on the inside forms. After being leveled and plumbed, the forms are braced and held in position. The outside wales are next bolted together.

The location of the jack clamps is now laid out on both the interior and the exterior wales and the wales are bored for the ¾-in. round lifting rods. The jack clamps are then set into place, plumbed both ways, and held in place by toe-nailing them to the wales. In setting the jack clamps the center line of the clamps must coincide with the center line of the walls to be concreted. Where inside and outside wales are not the same width it is necessary to notch the wider wale for the clamp post so that the clamp will center on the wall.

The jacks are next set in a central position on the top crosspieces of the jack clamp and are lag screwed down to the crosspieces. The lifting rods, which are ¾-in. round steel threaded 3 in. on each end, are then bolted into place as shown on Fig. 1. All wales should now be cleated together at the flexible joints directly under the jack clamp posts. These cleats may be either nailed or bolted, using not too many nails or bolts, as the object is to hold the wales together rather than to make one rigid piece of the several wales.

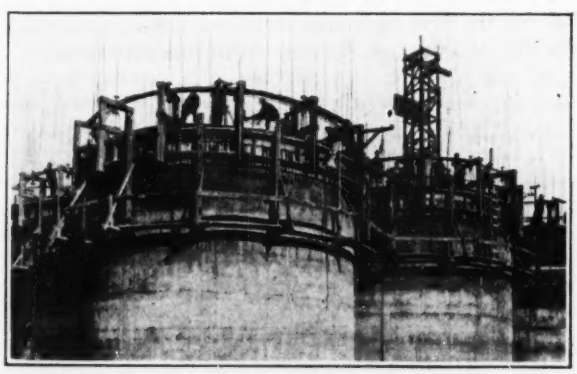


FIG. 3—MOVING FORMS FORTY FEET UP ON CYLINDRICAL BINS

A 1x12-in. carrying board as shown in Figs. 6 and 3 is next tacked into place directly below the top wale on the outside forms. This carrying board breaks joints where the wales break joints. Then 2x6-in. braces are nailed into place on both sides of all jack clamp posts to transmit the lift from the lower wale to the top wale. These braces are used on both inside and outside forms. Where the brace and the carrying board both occur, the brace is notched out for the carrying board.

As the initial fill in the sliding forms must be to a depth of approximately 3 ft. the lower portions of the forms are subjected to a strain at this filling which is greater than they will be subjected to in any later operation. This makes it necessary to provide special bracing for the bottom of the forms for this fill. It can most easily be done by a plank nailed between the bottom of the 1x6-in. staves and the lower extremity of the jack clamp posts, and for the inside forms by the use of cross bracing from one form to another. The temporary bracing, in addition to holding the

forms true, will close up any openings through which the concrete might leak at the bottom of the forms.

The forms are now ready for the initial fill, but to facilitate the work throughout the job it is necessary to have an operating platform from which to work and over which the concrete can be wheeled. This platform may be built of 1x6-in. D. & M. roofers laid on 2x6-in., 2x8-in., or 2x10-in. joists. The ends of these joists rest on the top wales of the inside form as shown in Fig. 1.

Before concreting is started it is best to have the first length of all vertical wall and column reinforcing steel in place. To hold this steel in position, set it in templets placed about 7 ft. above the level of the working platform. Uprights 2x4 in. are nailed to the outside posts of the jack clamps, and templets are supported from these uprights. All vertical wall steel is next placed.

Wherever possible place all horizontal reinforcing steel for the first 4 ft. of wall before the outside members of the sliding forms are placed. To do this, vertical dowels of $\frac{3}{8}$-in. or $\frac{1}{2}$-in. steel must be set into the foundation slab on the center line of the walls to be built. These dowels should be set on 4-ft. to 5-ft.

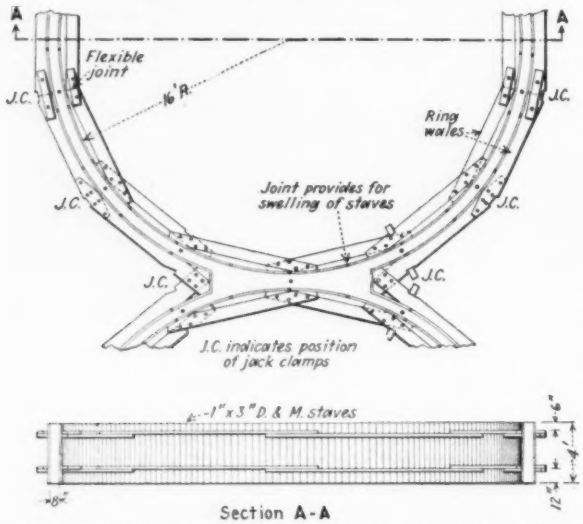


FIG. 4—PLAN AND SECTION OF CIRCULAR WALL FORMS
Location of jack clamps indicated by the letters J.C.

centers and should extend up 4 ft. With these dowels it is possible securely to wire all horizontal reinforcing steel for the first 4 ft. in height to them. The use of these dowels, and the wiring of the horizontal steel to them, has a further decided advantage as it anchors the initial fill of concrete to the foundation slab and lessens the possibility of the concrete lifting with the forms when jacking is started. Fig. 2 shows the horizontal steel in place for the first 4 ft. of fill before the exterior forms are in place.

First Fill—When the forms are ready, concrete is poured to a depth of 3 ft., all walls being filled evenly. Spreaders must be kept in the top of the forms throughout this operation to prevent the pressure in the bottom of the form from partially collapsing the forms at the top. While the initial fill is being placed and prior to the concrete getting any set, 1-in. round jack rods are dropped into place through the jack screws. These rods must have end bearing on the foundation slab. In order that only a portion of the jacks may run out of the jack rods at one time as

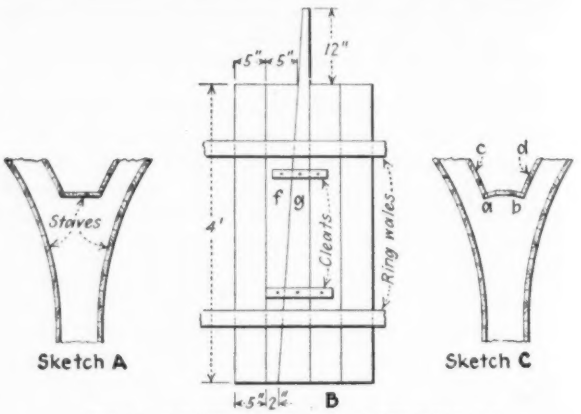


FIG. 5—ADJUSTMENT FOR SWELLING AT JUNCTIONS OF BINS

the forms are jacked up, various lengths of jack rods must be used at starting. While the concrete is still soft the alignment of all forms should be carefully checked and if any are found to be out of alignment they should be braced back into position.

The concrete in the forms must be watched very carefully for set, as it is in jacking loose from the first fill that there is the greatest danger of lifting the concrete. The bottom bracing is removed and the jacks are turned slowly just as soon as the bottom concrete has set sufficiently to hold its shape without slumping or running. At this point both the forms and the concrete must be watched to be sure that the forms are sliding and that the concrete is not slumping away below the forms. Concreting can be resumed and the forms filled to the top just as soon as it is seen that the forms are sliding properly. While the forms are being filled to the top they must be kept moving upward. The speed at which they are lifted is governed by the setting of the concrete.

When the first fill of sliding forms is made in cold weather, making artificial heat necessary, the application of the heat must be thoroughly studied. The tendency is for the heat to be most effective about a foot above the base of the forms and away from the cold slab on which the forms set. Then the concrete 1 ft. above the bottom will set sufficiently to adhere firmly to the forms while the concrete in the bottom will still run and delay starting the jacking until the concrete a foot above the bottom lifts and breaks when jacking is started.

Daily Fills—After the forms have been filled to the top for the first time and have been jacked loose from the initial fill of 3 ft., the entire operation may be

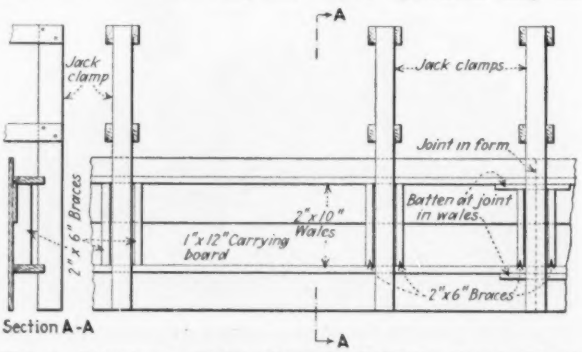


FIG. 6—EXTERIOR ELEVATION OF SLIDING WALL FORM

made continuous by the use of either three 8-hour shifts or two 8-hour shifts with 12-hour shifts on the jacking, or it may be run on a single 8-hour shift with an 11-hour to 14-hour shift on the jacking. When

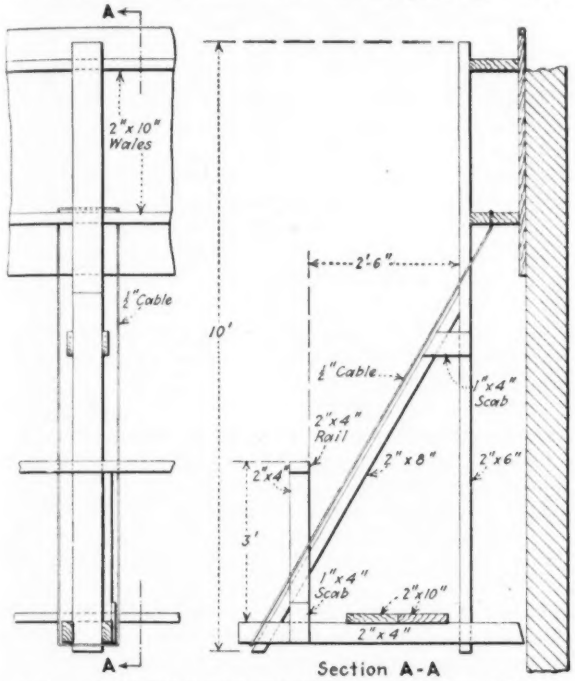


FIG. 7—SCAFFOLD FOR FINISHING CONCRETE BELOW SLIDING FORMS

operated on a single 8-hour shift the forms are left full at the end of the shift and the jackers continue jacking very slowly for 3 to 6 hours until the forms are jacked clear of the concrete. By clear of the concrete is meant that a slight opening from $\frac{1}{16}$ in. to $\frac{1}{8}$ in. exists between the concrete wall surface and the surface of the form. Four-foot forms should be clear of the concrete when the top of the form has been jacked up from 10 to 12 in. above the top of the concrete. The speed of jacking after the 8-hour shift of concreting has ended must be gaged by the setting of the concrete. Forms should never be jacked more than 13 or 14 in. above the top of the concrete. When the single 8-hour shift is used the top of the previous day's concrete should always be well grouted before concreting is started. At the start of the 8-hour shift the forms should be filled to the top and then maintained full throughout the entire shift while the jacking crew continues to jack throughout the shift.

A single 8-hour shift is very satisfactory in cold weather and can average from 4 to 6 ft. per day on a straight wall job where there are no doorways, windows, or beams to delay the work. In warm weather when the concrete sets rapidly a single 8-hour shift results in bad fill joints and two 8-hour or three 8-hour shifts are found much more satisfactory. In warm weather, using two or three shifts, an average daily fill of from 8 to 10 ft. can be obtained.

Mason's Scaffold—As soon as the forms have been lifted high enough to allow it, bracket hangers as shown in Fig. 7 are nailed to the wales on the forms. Scaffold plank are laid on these brackets to form a continuous hanging scaffold. These scaffolds are hung both inside and outside of the walls and from them the masons rub down the concrete surfaces as rapidly as they are exposed below the forms. As the concrete is quite green when first exposed the rubbing down can be done with a wood float.

Jack Rods—As the jacks are shifted up on the jack rods they come to the end of the rod and disengage. When this occurs the jacks are backed up above the end of the rod and a 1-in. pipe sleeve 6 in. long is inserted over the top of the rod. The next length of jack rod is then inserted through the screw piece of the jack and dropped into the pipe sleeve. The pipe sleeves are slightly dented at their center so that they will engage about 3 in. of the top of the first rod and 3 in. of the bottom of the second rod, thus forming a strong splice. The spring grip in the bottom block of the jack takes hold on the second jack rod and the jack is again ready to lift the form.

Swelling of Forms—It may happen through lack of care in providing for expansion in the sides of the interior forms that the form swells and binds too tightly against the concrete surfaces. Should this happen, the pressure can be relieved by removing one of the vertical staves and replacing it with a stave of narrower width. Should there be an excessive amount of binding, it may become necessary both to replace one of the wider staves with one of the narrower staves and to cut out a small section of the top and bottom ring wales at one of the flexible joints. After such a section has been cut out, the joint can again be scabbed over as before.

Checking Forms for Level—As it is essential to start the sliding forms out from a level position, so it is also essential to carry them up in a level position. If the forms are not jacked level the walls produced will

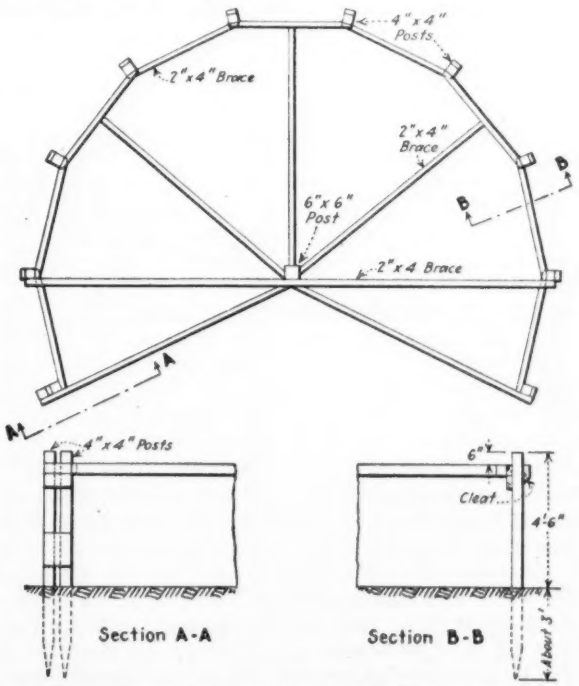


FIG. 8—FORM FOR BENDING REINFORCING RINGS

not be plumb. Forms should be checked for level about every 2 ft. of lift, and if found to be out of level at any time this should be remedied at once by jacking up at the low points. Daily inspection of the forms

and working parts should be made. This inspection must be thorough, covering all timbers, bolts, nuts, etc.

Bending Steel for Circular Bin Reinforcing—To bend the horizontal reinforcing steel into circular shape a bending rack as shown in Fig. 8 is erected. Tests must then be made to find the correct diameter to make this rack so that when the steel is bent around it and released it will be bent to the proper radius. This can be ascertained only by trial as all steel bars will spring out after being bent around the rack and this spring will vary for different grades and for different diameters of steel. Fig. 9 shows stakes driven in the ground, the inner circle representing the inner surface of the bin walls and the outer circle representing the outer surface of the walls. These stakes are used as a templet to try the steel for proper curvature. The steel bars must be bent so that they lie centrally between the two rows of stakes. After being bent the bars are racked in tiers ready for hoisting to the work.

Flat steel as ½x2 in. is now quite generally used for circular reinforcing rods. It need not be bent before being placed, but can be sprung into the circular shape provided the diameter of the bins is not too small.

Placing Reinforcing Bars—All reinforcing steel, with the exception of that in the lower 4 ft., is placed as the concrete progresses. As the lower 2x6-in. cross-braces on the jack clamps are very close to the forms it is impossible to place horizontal steel in advance of the concrete. Steel placing on a sliding form job then requires the constant careful supervision of a good foreman as all horizontal steel is lost to view almost as quickly as it is placed. Unless the work is carefully laid out and the plans are closely followed there is danger of getting more or less steel than is specified. Possibly the best check on the placing of the horizontal

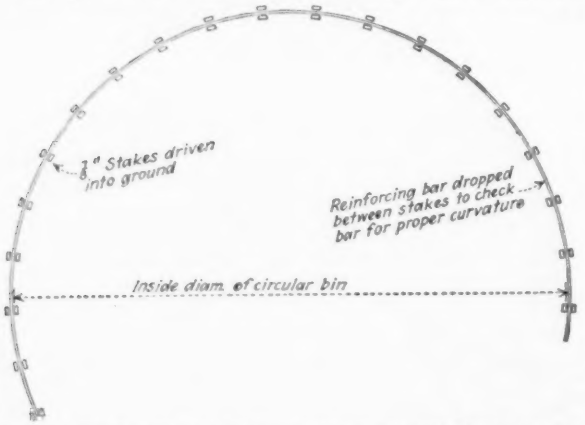

⅛" Stakes driven into ground
Reinforcing bar dropped between stakes to check bar for proper curvature
Inside diam. of circular bin

FIG. 9—TEMPLET FOR CALIBRATING REINFORCING RINGS

bars is obtained as follows: Distribute a definite number of bars around the forms on the working deck at one time. Keep a record of this number and a record of the required total number for each foot in height of the walls, and then record the elevation at which this steel should all be used up. By checking up to see that each lot of steel is used up at the proper elevation it will be definitely known that the proper average spacing is being obtained. Another way is so to gage the speed of the concrete crew that all walls are filled on each circuit a depth equal to the spacing of the horizontal reinforcing bars. This enables the reinforcing crew to follow the concrete crew with a single layer of reinforcing on each circuit of the walls.

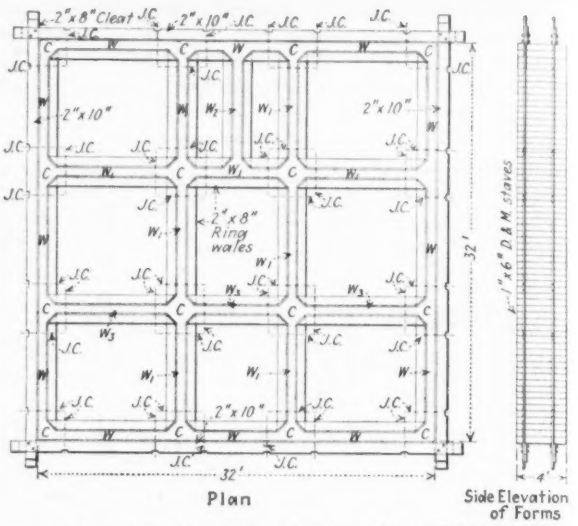


FIG. 10—SLIDING WALL, COLUMN AND BEAM FORMS

Column forms marked *C* concreted in all stories. Wall forms marked *W* concreted in all stories. Wall forms marked W_1 concreted in 5th story only and used as form to concrete beams for 2nd, 3rd, 4th, 5th floors and roof monolithic with adjacent columns. Wall form marked W_2 concreted in 5th story only. Wall forms marked W_3 concreted in 1st, 2nd, 3rd, and 5th stories and used as form to concrete beams for 5th floor and roof monolithic with adjacent columns. Location of jack clamps is indicated by J.C.

It is essential that the reinforcing bars be kept at least 1½ in. from the outside surfaces of the walls. With stationary forms this can be accomplished by the use of a concrete spreader placed between the steel and the forms, but this is not possible in the case of sliding forms as the forms will catch on the spreader and tear the concrete. Extreme care in placing the reinforcing bars is the only safeguard against having the steel too close to the exterior surface.

Organization and Operation of Jacking Crew—The essential point is so to organize the crew that the forms will all be lifted at the same time and will be lifted uniformly in height. On the ordinary operation where forms are lifted approximately 4 ft. in 8 or 9 hours one man can do the jacking on 12 to 16 jacks. Each man's work, however, must be limited to certain jacks so as to prevent any possibility of duplication. The crew is organized with a definite group of jacks assigned to each man. The foreman of the crew usually stands where he can see every man in the crew. At a signal by the foreman every man in the crew starts jacking by turning each jack in his group a half turn or 180 deg. A definite rotation is followed by each man which insures against his failing to turn any one of the jacks, or the possibility of his turning any one of the jacks in his group twice, on the same circuit. The foreman watches his crew, and as soon as the last man has completed, the whistle is again sounded to give the men the signal to start over their circuit again. A supply of jack rods is kept on the working deck so that as the jacks run out of the rods new rods can be inserted by the jacking crew.

Exterior Finish—All exposed concrete surfaces are finished with a wood float as rapidly as these surfaces are exposed below the sliding forms. All voids are filled and all board marks or irregularities are rubbed out in this operation. This work can be done at a small cost per square foot while the concrete is still green, but the cost will be excessive if the rubbing is left to be done at some later date when the concrete is hard.

In the case of grain bins, this damp-proofing can best be accomplished by two coats of a good concrete paint applied to the exterior surfaces of the bin walls. This also greatly improves the appearance of the bins and prevents moisture from getting through to the reinforcng steel.

In case the bins are painted the painting should not be done until the concrete has thoroughly cured. In general by the time the concrete bin walls have properly cured the surface has become covered with dust or soot. To make sure that the paint will stick, the wall surfaces should be brushed with wire brushes.

Selection of Plant and Equipment—The governing factor for size of mixer to be used is the quantity of concrete necessary to pour all of the walls for a height of 4 or 5 ft. in 8 hours. The mixer should have a daily output capacity somewhat greater than the daily yardage to be poured. A bucket hoist will be found most economical. As the working platform, however, is being constantly elevated some arrangement by which the concrete hopper on the tower can be easily raised, is essential. The best is a steel concrete hopper which is attached to a steel sliding frame which can be raised or lowered on the tower by means of a cable and winch. By using a short chute from the tower hopper to the buggies into which the concrete is deposited for distribution to the forms, it is only necessary to raise the tower hopper and sliding frame about once in 8 hours.

Distribution of the concrete on the working platform to the forms is best accomplished by the use of two-

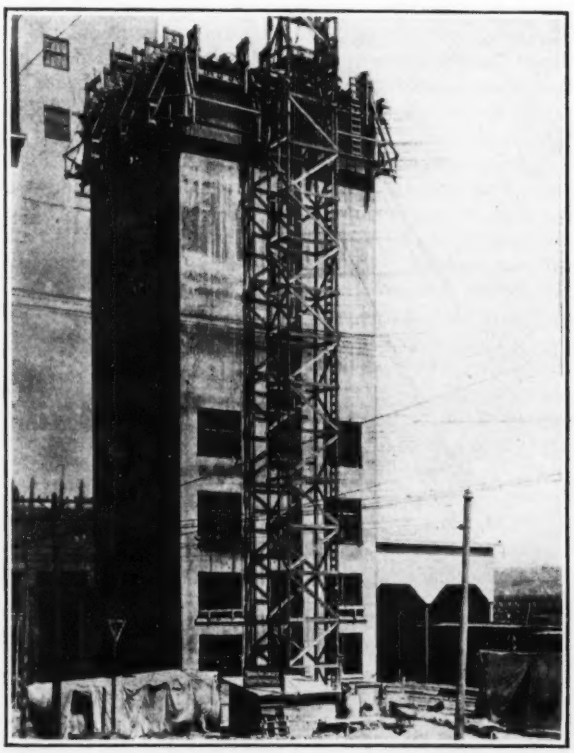


FIG. 11—BUILDING WITH COLUMNS AND FLOOR GIRDERS CONCRETED WITH SLIDING FORMS

wheel concrete buggies. As a buggy full of concrete will weigh approximately 700 lb. it is necessary that the working deck be strong enough to support a rolling load of this weight. It will be found economical to lay runways on top of the working platform on which to run the buggies.

As the reinforcing steel, jack rods, and jack clamps project a considerable distance above the forms, spout-

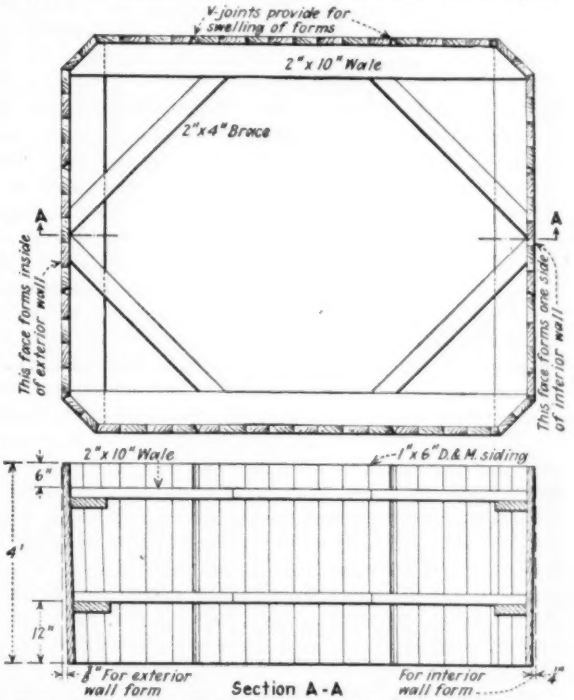


FIG. 12—INSIDE WALL FORM FOR BUILDING SHOWN BY FIG. 11

ing concrete into sliding forms is not practicable. It might, however, be possible to spout the concrete to the center of the working decks and to run it into the wall forms with chutes.

Choice of Aggregates—Choice of the aggregates for concrete to be poured into sliding forms is of great importance in the smooth working of the forms and in the surface finish of the concrete walls. Crushed stone, crushed gravel, or uncrushed gravel may be used for the coarse aggregate, but all of this material must be not over 1 in. in diameter. Larger stones may arch or wedge in 6-in. or 8-in. walls and will roll due to the movement of the forms. The sand should be well graded from small particles to particles up to ¼ in. in diameter. A uniform size sharp sand of large grains is very undesirable as it will cut into the surface of the sliding forms and will roll on the forms resulting in a surface of very coarse texture. The labor involved in rubbing the concrete to a uniform even surface will be materially less when the proper sand is used in the mixture. Crushed stone screenings or dust are frequently used as the sand aggregate in concrete. This material is, however, not well suited for concrete placed in sliding forms as it mixes poorly with the other aggregates used and produces a mortar which works hard and tears easily.

Concrete Mix for Sliding Forms—The mix for sliding formwork is usually specified as 1:2:4. This is satisfactory when the proper sand is to be had, but generally it will be found that the proportion of stone will have to be reduced in order to produce concrete of the proper consistency. To obtain good results in sliding formwork the concrete must be placed much

drier than in ordinary stationary formwork. Any excess water in the concrete will run away leaving rough stony patches in the concrete and must therefore be avoided. The excess mortar required in the concrete for sliding formwork is larger than that required in stationary formwork. In order to facilitate the work a uniform mixture of concrete throughout the day's work is essential. When this is had every portion of the concrete in the day's work will be at the same stage of set at a given level at any one time.

Sliding Forms in Structures with Columns and Floors—To illustrate the use of sliding forms for rectangular buildings with exterior and interior walls and columns and with floors, a description of the erection of a grain drying building for the Wyandotte Elevator Co. at Kansas City, Mo., is given. The general design of the forms and the construction operations are identical with those for circular bins.

The Wyandotte building is 32x32 ft. in plan and 88 ft. high above the first floor. Sliding forms were used to construct all walls, columns, and floorbeams from the first floor to the roof. Five intermediate floors occur between the first floor and the roof. The interior sliding wall forms were used to form all interior columns, walls, and floorbeams, and were carried up with the exterior wall and columns forms. All floor slabs were placed afterward on top of the floorbeams, the concrete being spouted from the operating deck of the sliding forms to the slab forms below. Fig. 10 shows a plan of the building. Fig. 11 shows the sliding forms at roof level.

The forms in general were similar to those for circu-

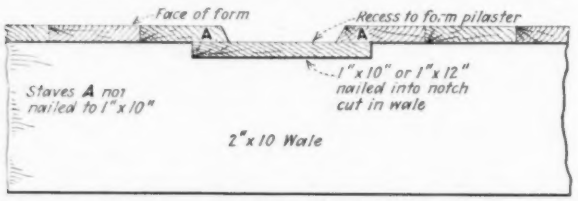


FIG. 13—PROVISION FOR SWELLING

lar bins except that they were square. Fig. 12 shows the inside form. In cases where a small offset or pilaster in the face of the walls is not objectionable provision in the forms for swelling can be made as shown in Fig. 13 to obtain even better results than are obtained from the V-joints in the staves. Fig. 14 shows one unit of the outside forms. These are made with 2x10-in. wales placed at the same relative height as those on the inside forms. Outside forms for rectangular buildings are made in sections varying in length from 12 ft. to 16 ft. and so figured that the joists between sections come directly under the vertical posts of the jack clamps.

The procedure in placing the forms for the first fill is identical with that for circular forms. It is essential in this case to have the wales on the exterior forms extend approximately 2 ft. beyond the building lines in order to have sufficient stock to cleat or bolt the wales on adjacent sides of the building firmly so as to hold the corner true. The 1x12-in. carrying board as shown in Fig. 6 should be used in all cases with rectangular forms. It distributes stresses in forms due to jacking and makes for better alignment of the forms. In the case of rectangular forms the use of the carrying board will allow a greater spacing of jack clamps.

Door and window openings, Fig. 11, were built by inserting wood frames of the proper size into the top of the sliding forms, anchoring these frames down to

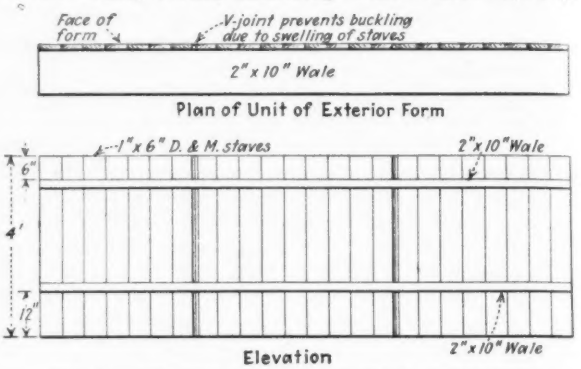


FIG. 14—SECTION OF OUTSIDE WALL FORM FOR RECTANGULAR BUILDING

some of the vertical steel, and then sliding the forms by these frames.

From Fig. 10 it will be noted that forms as originally placed for the Wyandotte job provided for four interior columns with two interior cross walls in each direction. The cross walls occur only in the fifth story, but beams the same width as the walls occur at every floor level. In the building up of the forms the columns were produced by hanging a stopper in each wall form adjacent to the columns. These stoppers slide up with the forms. As the top of forms arrived at top of beam level a beam bottom form was placed inside of the interior wall forms at the proper elevation. These beam bottoms were shored from below with 4x4-in. shores and the stoppers in the wall forms at columns were driven down to the level of the beam bottom. The reinforcing was then placed in the beam forms and the beams were concreted their entire depth. All of this work caused a short cessation in the jacking, but as soon as the beams were filled jacking was resumed, new stoppers placed in the interior wall forms to stop off the column concrete, and the forms jacked past the beams just as though they were walls.

Where jack rods carry up outside of the concrete wall or in clear stories between beams, it is necessary to brace these rods at intervals of approximately 4 or 5 ft. in height to prevent their bowing. A bowed or bent rod is of no value in lifting a load. Where jack rods go through beams and extend into the clear story they must be sawed off later just below and just above the beams. Some labor and some steel may be saved, however, if a sleeve is placed on the jack rod where it passes through the beam. When this is done the jack rod may be slid out of the beam and the entire rod can be reclaimed.

In the past sliding forms have been used but very little in the construction of buildings containing floors and columns, yet on certain types and layouts of buildings their use would not only produce a very satisfactory building but would also be the means of producing greater speed in construction and greater economy. This is especially true in the case of tall buildings having a large wall area, either inside or outside, which is typical in a number of stories. For buildings with a number of floors the layout of the beams must be very nearly typical for all floors if sliding forms are to be used in their construction.

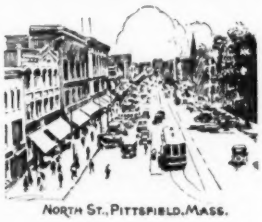
Engineering in the Small City

A series of notes from the field on the Problems and Practices of the Municipal Engineer in the Cities Below 100,000

The Editor Visits the City Engineer

Snapshots of Our Representative Small Cities

Pittsfield, Mass.



NORTH ST., PITTSFIELD, MASS.

IN BERKSHIRE COUNTY, MASSACHUSETTS, 150 miles west of Boston. Pittsfield is a city, including several villages within its corporate limits. Its site, a hill-girt valley in the Berkshires, makes it a popular summer resort. The area is 41 square miles, and the city has 150 miles of streets. The leading manufactories are woolen and silk mills, foundry and machine shops, electrical machinery and supply works, clothing shops, a piano factory, paper mills and paper mill machinery factories. Settled in 1743 as Pontoosuck, or Boston Plantation; incorporated as town, under present name, in 1761; chartered as city in 1891. Pop. (1920) 42,000.

MUNICIPAL affairs in Pittsfield are presided over by a mayor and city council, the latter composed of a board of aldermen and the common council. The members of the city council in joint ballot elect most of the city officers. The city engineer owes his appointment to the Board of Public Works, whose direction and control of the public works of the city include the water-works. Arthur B. Farnham has been engineer since 1906. The position is now under the civil service law. In addition to the engineer, the Board of Public Works contains, beside its three members, a clerk, two water inspectors, a superintendent of streets, a superintendent of water-works, a foreman of sewers, and two office assistants. Though with such an organization the engineer's position could well be subordinated, in few cities visited in connection with the present study is there a greater consciousness on the part of general municipal administrators of the need for competent engineering services and adequate structures than at Pittsfield.

Street Work—About 25 years ago practically all of the new paving being laid in Pittsfield was sheet asphalt. However, on the principal street of the city repair gangs appeared and frightened the populace into believing that continual patching would be the result of laying any great amount of sheet asphalt. Fifteen years ago the city started on brick and some asphalt macadam, and then the cement people did some effective educational work, and since that time cement-concrete has been the prevailing type. The contractors are thoroughly equipped to lay that type of paving and are used to putting it down in efficient fashion, so it is likely there will be little change from cement-concrete for some time. The concrete is laid 7 in. thick with a longitudinal center joint and transverse joints spaced about 60 ft. Expansion joints ¾ in. thick are placed between the road and the curb and are hard enough for the screed boards to ride on so that there is considerable economy in rolling, belting, and hand floating. The surface of concrete roads on grades exceeding 5 per cent is roughened. The same contractors who do city work do a great deal of state work and the city engineer reports that the construction is generally excellent. A standard mix of 1:2:3½ is used. Until last year trap rock was shipped 40 miles and used, but now washed crushed gravel, which is locally available, is used generally. It is an excellent quality of quartzite. The pavement is costing less than $3 per square yard, exclusive of the cost of the steel.

As there are numerous dirt roads treatment given them during the summer months, particularly, is extensive. The property owners are given the option of choosing water or oil for dust laying, and the controlling opinion along any particular street rules. Sprinkling and oiling are generally paid for on the same basis—6c. per front foot. Roads of this character are first worked with a grader and the ditches and culverts cleaned.

Cement curbing presents departure from the usual practice—use of granite block. All new curbs in Pittsfield are cast-in-place cement curbs, 2 ft. deep, 8 in. wide at the top and 10 in. at the bottom. They are of plain concrete of a 1:3:5 mix. There is some doubt but that a richer mix might easily be used. The same mix is used in sidewalk work. The only departure from this standard curbing is on curves at street intersections, where the curb is of granite 8 in. wide and 12 in. deep and peen-hammered on the face and top. It is set on a 6-in. concrete base, which strengthens and backs it. The cement-concrete curb costs, including all excavation, about $1.50 per lineal foot which is practically $1 per lineal foot less than the price for granite. All sidewalks are of cement 5 in. thick including a 1-in. mortar finish. Sidewalks are paid for half by the city and half by the abutting property. Abutting property pays 40 per cent of the total cost of paving, including all incidental work, the other 60 per cent being paid by the city.

A most unusual procedure is followed by Pittsfield in laying pavement along car-line streets. The street railway company, pleading poverty, has been exempted for a number of years from payment of any pavement cost between rails. Therefore, all the paving is done at once and paid for by the city for the full street width. While this mileage is not extensive, it increases considerably the cost of paving car-line streets.

Yellow is used generally for painting traffic control lines at street intersections and around parking areas. Water hydrants are painted yellow with some phosphorescent silver paint applied to their tops. This greatly increases night visibility.

Water and Sewerage Construction and Financing—The city gets its water supply from a number of lakes and brooks a few miles outside the city limits. The principal impounding reservoir is that known as Farnham reservoir which was built in 1910. At that time an additional supply was needed. Mr. Farnham made the original engineering investigation and turned over his data and recommendations to a committee on water supply, composed of five leading citizens; this committee performed its work with extraordinary thoroughness. It engaged the late Hiram A. Miller, then chief

engineer of the Charles River Basin Commission and for years with the Massachusetts Metropolitan Water and Sewerage District, to go over Mr. Farnham's recommendations. Mr. Miller directed the design of the impounding dam which stores some 400 m.g. of water. Prof. Burr checked the design. Thus, most competent engineering advice was secured before construction started. During the construction of the dam Mr. Miller was the consulting and chief engineer of the project and Mr. Farnham was principal assistant engineer. That construction cost in the neighborhood of $1,000,000. The total storage capacity in the Farnham reservoir, according to the 1924 report of the Board of Public Works, amounted to 516 m.g. with flashboards in place, and in the Ashley Lake reservoir to 450 m.g. with flashboards in place. Even then, 1924 and 1923 being extraordinarily dry seasons, the supply was greatly taxed.

In the payment for water-works extensions the city has to be guaranteed 6 per cent on the investment made. If the income is less than 6 per cent of the amount of money involved in making the extensions, the property owners are assessed for the deficit. The abuttors who request the extension of water service have to sign a bond guaranteeing this 6 per cent. For instance, if a $2,000 improvement is made it must yield $120 interest the first year. There is some agitation in favor of direct assessments for water extensions, but so far nothing has been done.

The sanitary and storm-water sewerage systems are separate. Sanitary sewage is collected at the pumping station and pumped to sand filtration beds about 2 miles from the city. The surface water drains easily, the topography of the city allowing it to be led to branches of the Housatonic River. Payments for sewer extensions are based, first, on direct assessment of 25c. per front foot of the property benefited, and on an additional tax of 4 mills on the valuation of the property benefited per square foot over an area extending 100 ft. from the street line. This amounts to about 65c. to a front foot each side of the street. Naturally, if excavation is heavy and construction extensive, the city is the loser by such an extension. However, it takes the gamble. If all of the sewer extension assessment is not paid at once by the abutting property owner, that sewer construction amounts to a lien on the property. However, the assessment is made only once and that upon the valuation on April 1 of the year the sewer is put in. The city engineer makes sewer assessment maps during winter.

The finances for construction and maintenance of public works are not secured from the same source. Maintenance funds are taken from the general budget. New work of a permanent character is paid for usually by bond issue. No matter what improvement is needed, if it is desired to issue bonds, then a sum representing 25c. of each $1,000 of the valuation of last year must be appropriated from the tax levy and the remainder of the sum to be appropriated from the issue of bonds. No bond issue can be made in a less sum, which in 1925 amounted to $13,173. Any improvement, therefore, costing less than that would have to be paid for out of the general fund. This places discussion of smaller improvements directly before members of the council. It also acts as a curb on extensive issuance of bonds.

There are 44 regular bridges owned and maintained by the city. Most of them are short-length bridges but require a certain amount of maintenance. The tendency in bridge reconstruction now is toward reinforced-concrete arches, so that maintenance can be reduced to a minimum. The principal maintenance required is bridge floor replacement. As fast as this is done, old wooden floors are discarded in favor of semi-permanent or permanent construction. The classification of the bridges is as follows: Steel truss with wooden floor, 7; iron truss with wooden floor, 2; steel girder with permanent floor, 3; steel girder with wooden floor, 1; iron bowstring with wooden floor, 4; I-beam with permanent floor, 12; I-beam with wooden floor, 9; wood beam with wooden floor, 2; and reinforced-concrete arch bridges, 4. The I-beam bridges, which predominate in number, range from 20 to 35 ft. in span.

Engineering Work

Intercepting Sewers and Disposal Plant at Laramie, Wyoming

By F. M. Veatch
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THE city of Laramie, Wyoming, has recently completed an intercepting sewer which with a new outfall sewer and a disposal plant has given relief from serious sewer surcharging in the business district, made the old sewer system adequate, by limiting its tributary

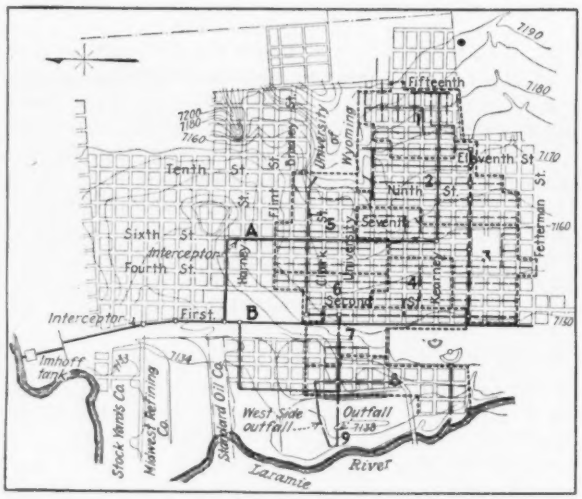

FIG. 1—SEWER SYSTEM OF LARAMIE, WYO.

area, and has given the city the backbone of a larger system which can be built little by little as the growth of the city requires. Laramie has a present population of approximately 7,500 and is an important division point on the Union Pacific R.R. Industries are represented by extensive stockyards, three plaster mills, a large oil refinery and railroad shops. The University of Wyoming is also located there and represents a non-resident population of approximately 2,500 persons. A unique feature of this project is the existence, in Laramie, of a gravity water supply fed by mountain springs and with no water charge or regulations as to use further than those enforced by nature in the form of pipe line capacity and available water, which condition fosters excessive use of water and explains the high observed sewage flow of 264 gal. per capita.

Of the Small City

Investigation—In order to get definite information in regard to the sewage flow, the system was divided into sections as shown on Fig. 1, and gagings were made by means of weirs fitted in the manhole inverts. While gagings were being taken, a house to house canvass was made to determine the density of population and the number of sewer connections, and this information was tabulated on cards, indexed by blocks and street numbers as shown below. The information on the card is rather more detailed than was necessary but was obtained for use later in a water waste survey.

BLOCK.....ADD.....STREET, NO.....
Residents House .....No. Rooms.....
Employees in Building .....

	Yes	No	Leaks	Remarks
WATER				
SEWER				
YARD HYDT.				
LAUNDRY				
TOILETS				
SINKS				
LAVATORY				
BATH TUBS				

The canvass was made by ten university students in three days, which time included revisits to approximately 5 per cent of the houses. The results of the canvass and the sewer gagings are given in the following table:

AREAS OF VARIOUS SEWER DISTRICTS, POPULATION DENSITY AND PER CAPITA SEWAGE FLOW AT LARAMIE

Dist.	Area, Acres	Tributary Population	Density Persons per Acre	Sewage Flow Max-Gal.	Min. per Capita	Avg. per Day	Per Cent Sewered
1	64.5	460	7.14	95	53	75.2	97
2	99.5	1,842	18.50	143	74	97	93
3	106.5	3,167	29.78	136	97	114	88.5
4	36.1	438	12.18	430	258	420	94
5	90.5	1,005	11.10	271	148	188	94.5
6	34.3	1,404	4.09	692	189	341	94
7	56.0	491	8.77	.....	...	447	....
8	63.7	822	12.82	91	68	83	59.8
9	551.1	6,321	11.47	337.8	182	264	86.7

The need for relief was evident in the sewers, which served the developed residence districts (1, 2 and 3) and the business districts (4 and 6). Therefore the intercepter was designed with two branches, one (*A*, Fig. 1) to the southeast to intercept enough of districts 1, 2 and 3 to make the old system adequate, and one (*B*, Fig. 1) to the west part of town to intercept the old outfall sewer, and to intercept the remainder of the sewage from districts 1, 2, and 3, above its confluence with that from the other districts. The two branches join in the northern part of the city as shown in Fig. 1, and the main intercepter extends north approximately 3,620 ft. to the disposal plant.

It was assumed that the future population density over the entire district to be served by the new sewers would not exceed 12 per acre and that the sewage flow would not exceed 250 gal. per capita since the growth of Laramie will probably be in the nature of expansion rather than crowding, also since the present wasteful use of water is too expensive to be continued indefinitely. The outfall sewer was therefore designed on a basis of 0.0046 sec.-ft. per acre, the sewers to flow half full.

Disposal Plant—The minimum flow of Laramie River is approximately 30 sec.-ft. and probably nothing further than local nuisance would result from the direct discharge of sewage into the stream. However, it was decided to give the sewage preliminary treatment to satisfy the owners of riparian property below the plant and a plant consisting of an Imhoff tank and sludge filters was designed. The Imhoff tank, Fig. 2, was designed to serve a population of 8,000 at 250 gal. per capita per day, and space was provided for a duplicate unit to be constructed later.

The settling compartments, four in number, were designed to give an average settling period of 1½ hours and the sludge compartment was designed on the basis of 2 cu.ft. per capita, to provide storage for the long periods necessary in this climate. The sludge compartment was constructed in the form of four hoppers each 6 ft. deep and 22x23 ft., with side slopes of 1 to 2.

FIG. 2—IMHOFF TANK AND SLUDGE BEDS AT LARAMIE

Lack of available head prevented the use of the conventional siphon method of sludge removal, since the level of the sludge bed is above that of the water level in the tank. Therefore, the sludge pipes from the four hoppers were connected to an engine-driven centrifugal pump located over the tank and arranged to discharge into a flume leading to the sludge bed.

Sludge Bed—A conventional sand bed for sludge drying with a capacity of 0.4 sq.ft. per capita was provided for sludge treatment and it is contemplated to take care of each year's production of sludge by three dryings made during the summer months. The contract for this work was let in March, 1923, to D. S. Reed of Littleton, Colo., and was completed in June, 1924. The investigations and preliminary work were done by Black & Veatch of Kansas City, Mo., in connection with E. K. Nelson and B. C. Bellamy, of Laramie. Plans were prepared by Black & Veatch, and the work was done under the supervision of E. K. Nelson.

Principles of Soil Mechanics: II—Compressive Strength of Clay

Modulus of Elasticity Determined from Cube Tests at Different Moisture Contents Shows Constant Ratio to Capillary Pressure—Poisson's Ratio for Clay—Analogies with Metals—The Iowa Experiments

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Temporary Lecturer at Massachusetts Institute of Technology, Cambridge

FOLLOWING the tests of confined samples of clay reported in the preceding article (*Engineering News-Record*, Nov. 5, 1925, p. 742), cubes of clay were tested by methods the same as used for ordinary solids. These tests brought somewhat different phenomena into view. In particular they threw light on resaturation action, on lateral effects, and on the analogies between clay and solids. The careful attention paid to the elastic properties of clay cubes was warranted by the fact that the behavior of clay cubes under load was found to be closely related to what is called the bearing capacity of clay deposits. As a consequence a clear understanding of the effect of load on clay cubes represents the key for understanding the more complicated effect of a load placed on the surface of a clay deposit.

Elastic Behavior of Clay

Cubes 2 cm. and 4 cm. on a side were used. They were molded within a lining of filter paper in a prismatic mold fitted with a plunger. According to the purpose of the test, the cube was either tested at once after molding or it was allowed to dry until the moisture content decreased to that desired. During the test the cube was surrounded by a tin cylinder inclosing an inner one of brass wire mesh, with watersoaked cotton between, to maintain an atmosphere saturated with water vapor. In such an inclosure the moisture content of the clay remained sensibly constant for several days, while a test lasted only two hours.

To measure the compressive shortening of the cubes measurements of great precision were required, and the accuracy of a micrometer screw with electric contact indicator proved to be insufficient. The difficulty was overcome by developing an interference contact indicator, consisting of a pair of thin glass sheets with a residual film of water between, a very sensitive micrometer screw, and an eyepiece. By noting the change in color of the Newtonian rings due to contact between the upper glass sheet (thickness 0.1 to 0.2 mm.) and the point of the micrometer screw, the position of the screw point can be determined to less than 0.00001 mm., which is far beyond the degree of accuracy of the micrometer screw.

Cube Test Results—The main diagrams in Fig. 1 may serve as an example of the results obtained by the tests. They represent the stress-strain diagrams for a set of cubes of yellow residual pottery clay with different percentages of moisture. The diagrams are very similar to compression diagrams of concrete and natural stone. Abscissas represent pressures (kg. / cm.^{2}), while ordinates give reduced strain, i.e., the ratio between the total compression and the reduced height—the height which the cubes would have if the volume of voids were reduced to zero at constant horizontal cross-section. Operating with reduced instead of ordinary strains is necessary in order to have a common basis for comparing with each other the strains produced by loads in cubes of different moisture content.

Strain Lag—In testing Cubes 1 and 2 the load was first gradually increased and then left constant for a certain time. During this time the strain increased, at decreasing rate (strain-time curves, under the stress-strain diagrams). This change of strain at constant load is unlike the change of moisture content at constant load in the tests previously described, as the latter process is due to the great resistance against the flow of water through the voids of the clay, while the former one goes on at constant moisture content, i.e., with the capillary water remaining in a state of equilibrium. The increase of the strain of clay cubes at a constant load seems to be identical with the elastic after-effects associated with the deformation of solid elastic bodies. The author has found that in clay it is due to the gradual compensation of unbalanced frictional resistances which at first take up a part of the load. Simple mathematical relations exist between the capillary pressure acting within the cube, the load and the value of the coefficients which determine the relation between the time and the increase of the strain.[1]

Cubes 3 and 4 in Fig. 1 were tested on a screw testing machine, in which the compression (strain) remains constant when the machine is stopped, while the pressure may change. As the lower diagrams show, when the compression is thus held constant the pressure decreases for a time. This phenomenon was found to be due to the same cause as the increase of strain at constant load, and it follows similar simple laws.

The more slowly the load was applied, the less marked were the elastic after-effects, but at the same time the steeper was the slope of the main branch of the stress-strain diagram. The main branch, which corresponds to infinitely slow application of the load, is called the reduced stress-strain curve (dash-line curves in Fig. 1). It can easily be constructed by means of the data furnished by the time-strain or the time-pressure curves.

Cyclic Loading—The next operation performed during the cube tests consisted in reducing the load to zero and re-applying it (complete cycle). The effect is represented by the hysteresis-loops. These loops differ from those shown in the pressure-moisture diagrams previously described by being straight instead of curved. When the load again reaches the value it had before release, the stress-strain curve asymptotically approaches the curve of continuous increase of load. Phenomena similar to these came in evidence when plotting the results of loading tests performed on the bottom of test pits in the field.

Modulus of Elasticity—The term "modulus of elasticity" should be confined to the reversible part of the deformation of the cube, represented by the hysteresis

loops. It is equal to the tangent of the angle between the strain axis and the axis of the hysteresis loop. In general this angle differs but very little from that of the initial tangent to the continuous-loading curve.

A very important relation was found to exist between the intensity of the capillary pressure and the value of the modulus of elasticity. The abscissas of the main branches of the pressure-moisture curves (see Fig. 3, p. 743, *Engineering News-Record*, Nov. 5, 1925) indicate the pressures required to reduce the moisture content of confined clay from its initial value (for zero pressure) down to the moisture content represented by the ordinates, provided surface tension is not active. It has been shown that the capillary pressure acting within a clay of definite moisture content is equal to the external pressure which in the pressure-moisture diagram corresponds to that moisture content. Using these pressure equivalents of the moisture content the curves in Fig. 2 were plotted. Here abscissas represent capillary pressures and ordinates represent moduli of elasticity. The values for Cubes 1, 2, 3 and a fourth cube intermediate in moisture content between 2 and 3 are plotted on the line marked "Yellow Pottery Clay." As the points mark out a straight line, it appears that the ratio between modulus of elasticity and capillary pressure of a given kind of clay is constant and independent of moisture content, provided that no resaturation of the sample has occurred. The lower line is the corresponding curve for a blue marine clay and the two upper lines for two sands. The simple relation which exists between the capillary pressure and the modulus of elasticity is of considerable practical importance inasmuch as it was found that the bearing capacity of a clay increases in simple proportion with its modulus of elasticity.

Analogy of Clay and Solids—Cohesion of clay depends upon the capillary pressure. In a similar way the cohesion of solid bodies is the result of the "intrinsic pressure," i.e., of the pressure per unit of area produced by the mutual attraction of the molecules. As the elastic properties of the clay cubes are very similar to those of solid cubes, the author suspected that the ratios between the moduli of elasticity of solids and their intrinsic pressures might also be a constant. This conclusion was found to be correct. In the upper diagram in Fig. 2, abscissas represent the intrinsic pressures of various metals (according to Traube, computed from thermodynamic data), while ordinates are moduli of elasticity for compression within rigid enclosures, computed from the ordinary moduli of elasticity and the

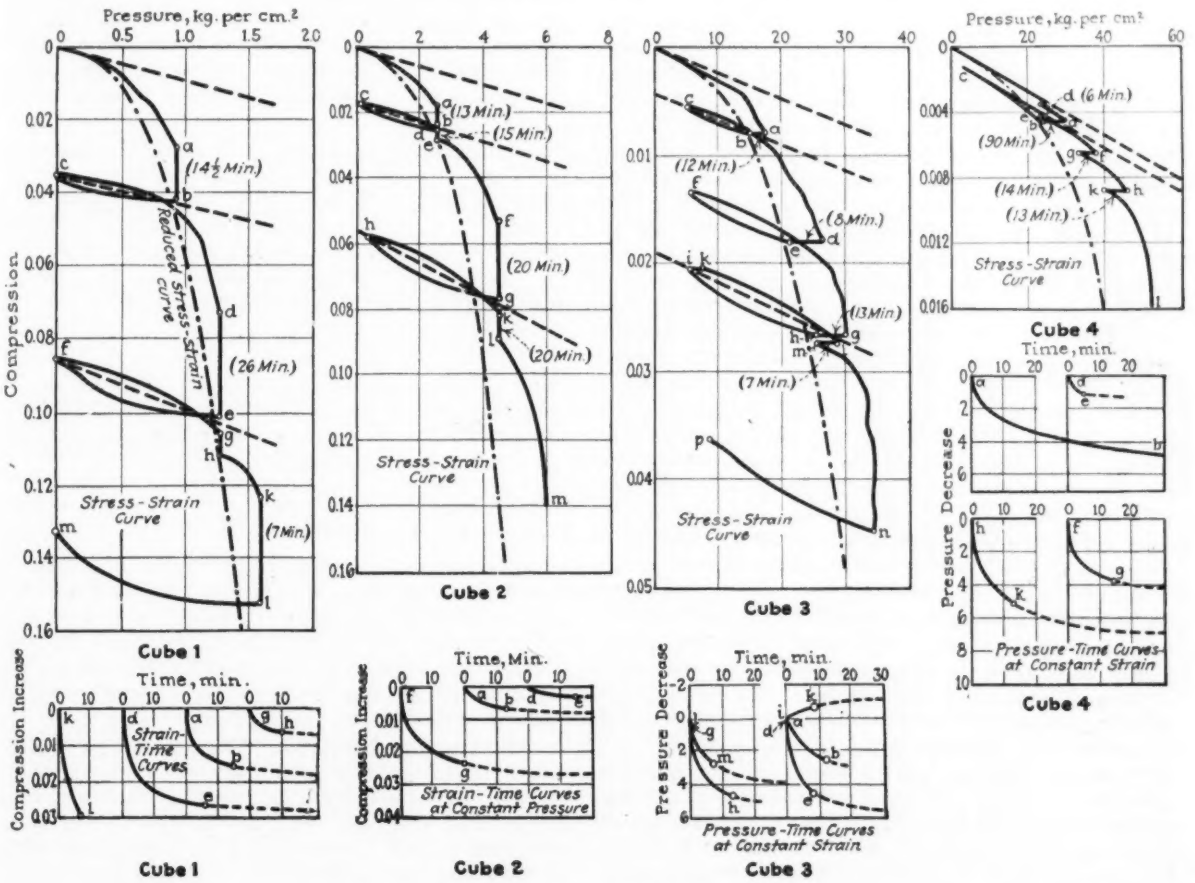

FIG. 1—COMPRESSION TESTS OF CLAY CUBES

Yellow residual pottery clay, specific gravity of solid matter 2.93, lower limit of plastic state 24.2, lower limit of liquid state 58.0 per cent, coefficient of plasticity 33.8.

	Cube 1	Cube 2	Cube 3	Cube 4
Moisture-Ratio e, (per cent of volume of solid matter)	0.792	0.681	0.490	0.482
Modulus of Elasticity (kg. per cm².)	115 to 76	310 to 195	3,760 to 3,460	7,300

Poisson constants. All the points are located along a straight line.

If the moduli of elasticity were perfectly constant for each clay cube, the hysteresis loops ought to be strictly parallel to each other, while in the diagrams Fig. 1 the slopes of the loops somewhat increase with the intensity of the pressure at which the cycle was started. In order to understand this we must keep in mind the fact that in case of a solid the intrinsic pressure remains constant during a compression test, while the volume decreases. It would be possible to maintain the volume constant by reducing the intrinsic pressure as the load is increased. In the case of a clay cube the volume cannot possibly change as long as the moisture content is unchanged, and therefore the capillary pressure decreases with increasing load, which in turn causes a decrease of the modulus of elasticity and a corresponding increase of the slope of the hysteresis loops. As a matter of fact, after having determined Poisson's ratio for several clays, the author was able to compute the relative position of their hysteresis loops, and the results checked fairly closely with the results of tests.

Poisson's Ratio; Resaturation—Poisson's ratio represents the ratio between the linear compression and the corresponding linear lateral expansion for a loaded cube. If one knows this ratio for any homogeneous material one can calculate the lateral pressure which the material will exert under load against a rigid enclosure. In turn, knowing the lateral pressure, one can compute Poisson's ratio. This method was used for the clays.

In order to find the lateral pressure exerted by a loaded layer of clay (mixture of clay and water, free from air and the clay surfaces covered with water so as to exclude the surface tension of the water), two sets of apparatus were used similar to the one represented by Fig. 2 (p. 743, *Engineering News-Record*, Nov. 5, 1925). The rings were larger and higher, and the clay was enclosed between two filters of equal dimensions and quality. At half the height of each layer of clay there was a horizontal steel tape, which passed out through slits in the enclosing ring. In one of the layers the flat side of the tape was horizontal, in the other one it was vertical. In order to keep the clay from being squeezed out through the slits, each slit was sealed on the inside by a small shield of thick filter paper. The ratio K between the horizontal and the vertical pressure acting within the clay is equal to the ratio between the forces required to overcome the friction between the clay and the two tapes. The frictional resistances were not measured until three or more days after the load was applied, to allow the hydrostatic stress differences to disappear. Computation from the results furnished the values of Poisson's ratio:

Yellow residual clay	(K = 0.70)	2.73
Blue marine clay	(K = 0.75)	2.55
(Water		2.00)
(Lead		2.24)
(Silver		2.63)

If both Poisson's ratio and the ratio between modulus of elasticity and capillary pressure are known, one can compute the reversible part of the deformation produced by a load on a layer of clay confined within a rigid ring. This reversible part is represented by the curved hysteresis loops of the pressure-moisture diagram (Fig. 3, p. 743, *Engineering News-Record*, Nov. 5, 1925). The computation was carried out for different kinds of clays. In every case the curve thus obtained coincided very nearly with the recurrent branches of the hysteresis loops of the corresponding pressure-moisture diagrams. This furnished a check on the computation of Poisson's ratio.

The recurrent branches of the hysteresis loops of the pressure-moisture-content diagrams are called the *resaturation curves*. They are simple logarithmic lines. The greater the resaturation coefficient, the greater is the increase in volume due to resaturation and the smaller is the ratio between modulus of elasticity and capillary pressure. The coefficient is greatest for clays rich in colloids. Experience seems to show that a clay is the more likely to slide the more it swells when brought into contact with water. Hence the value of the coefficient of resaturation may furnish a valuable indication of the stability of a natural clay deposit.

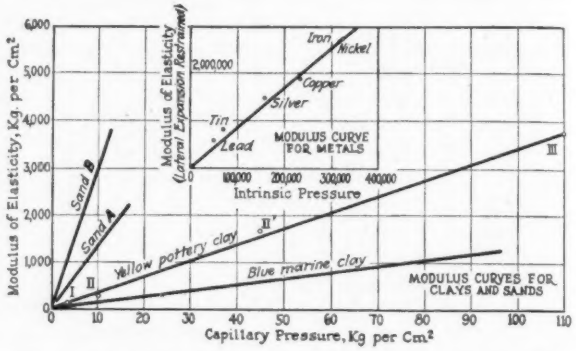

FIG. 2—RELATION BETWEEN MODULUS OF ELASTICITY AND CAPILLARY PRESSURE

Non-homogeneous Clay Soils—In 1921 and 1922 the engineering experiment station of Iowa State College determined the compressive, tensile and shearing strength of different kinds of clay exposed on the bottom of an old gravel pit[3]. The samples were taken from a depth of 1.5 to 1.8 m. below the bottom of the pit and were tested in undisturbed condition. The stress-strain diagrams seem to resemble closely those presented in Fig 1. Nevertheless, a thorough study of these diagrams leads to the conclusion that the two sets of tests differ in vital respects.

For the Iowa cubes the reversible part of the deformation is very small compared with the total deformation, and the high moduli of elasticity of the cubes seem quite out of proportion to their low compressive strengths. Calculation indicates unusually high coefficients of internal friction. One of the causes of these abnormalities seems to reside in the fact that the Iowa clay apparently had undergone repeated resaturation and shrinkage since its deposition, while the cubes of Fig. 1 were compacted under the influence of direct compression by capillary pressure. According to our experience, resaturation substantially affects the relation which exists between the moisture content and the capillary pressure. However, up to the present time, no exhaustive investigation of the effect of resaturation has yet been made.

The second and equally important cause of difference lies in the fact that the cubes of Fig. 1 were homogeneous, while the Iowa cubes were not. Thus, the paper

states, "The yellow clay . . . was not infrequently permeated by roots, wormholes and fibers or small crevices, and the strength would be correspondingly decreased." "One of the prominent factors appeared to be the friability or brittleness of the blue clay, as induced by the infiltration of water and air previously mentioned, which oxidizes the ferrous matter and causes the deposition of a brownish-yellow film along the planes of separation which honeycombed the structure. A slight shock as a result of this film or matrix was often sufficient to leave the blue clay a coarse granular mass" (p. 563). Clays which include planes of separation and seams of oxidized matter cannot be considered homogeneous; they represent an intermediate type between homogeneous clay and the crumb soils. In a homogeneous clay, the hydrostatic pressure of the capillary water is the same throughout the mass (provided the clay is in hydrostatic equilibrium); in a soil composed of crumbs the particles of each crumb are bound together by a very intense capillary pressure, while the crumbs themselves are as independent as are the individual grains of a mass of sand. As a matter of fact, accumulations of dry crumbs prove to have the same elastic properties as dry sands except for the fact that they are more compressible and less elastic than sands, on account of the brittleness of the crumbs. If water penetrates the voids of such a crumb mass, the crumbs absorb the water very slowly, because their permeability is small. Water flows from the voids between the crumbs towards the centers of important negative hydrostatic pressure, i. e. penetrates the crumbs. However, even after equilibrium is reached the soil represents a honeycombed mass which can by no means be compared to a homogeneous clay and if the water later on evaporates, the mass breaks up into crumbs again, on account of the non-uniform resistance of the material against the effect of secondary stresses.

FIG. 3—DETERMINING LIMIT OF LIQUID STATE (ATTERBERG)

Due to these facts there is a great difference between the elastic properties of crumb soils and of homogeneous clays. The former still need thorough experimental investigation. Fortunately the worst types of foundations (mud deposits, soft clays and the like) belong almost without exception to the homogeneous type of clay, whose physical properties are now known.

It is to be regretted that the Iowa investigations could not be supplemented by contemporaneous loading tests performed with the soil-testing apparatus proposed by the Foundation Committee.[3] The results would have furnished a most valuable contribution to our knowledge of the resistance of clays, because the relation which exists between the cube strength of a resaturated clay and the bearing capacity of the same material is not yet known.

Data Required for Describing Clays—Engineering literature contains hardly a single description of a clay that would allow us to identify the clay with those from other localities, or a description which gives us a clear notion of what the material was like. At best the describer contents himself with mentioning the color of the clay, its moisture content and its chemical composition. Experience, however, has shown that the chemical composition of a clay has but little to do with its physical properties, and, as for moisture content, some clays with 25 per cent of water are almost liquid while others containing 30 per cent are very stiff.

An exhaustive investigation of test boring samples requires too much time and labor to be justified except for scientific purposes. Study of the problem of investigating the properties of clays for *engineering purposes* has led the author to the conclusion that it would be advisable to modify the test program according to the character of the work affected. A detailed account of the data required for describing clays extracted from the sites of future floating or pile foundations, clays for dam construction purposes, clays which threaten to slide, etc., has already been published (in the author's book "Erdbaumechanik," Vienna, 1925). Some data however are required regardless of the special purpose of the description. These data concern the relation between consistency and moisture content.

The simplest and most reliable way of expressing this relation was devised by the late Prof. A. Atterberg, of Kalmar (Sweden), for agrogeological purposes, but it suits engineering purposes as well.[4] It requires a knowledge of the following data: Moisture content of the clay, specific gravity of the dry matter, lower limit of the plastic state and lower limit of the liquid state.

Moisture content is determined in the ordinary way. The sample is taken out of the test-pit or out of the well-boring auger and transferred into a wide-necked bottle with glass stopper. The bottle is filled completely so that there remains no air between the clay and the stopper, and the joint between the neck and the stopper is sealed with wax. In the laboratory a sample of the content is placed between two watchglasses and weighed, dried at 100 deg. C. and again weighed.

The specific gravity of the dry matter has to be determined by pycnometer. In order to get fairly accurate results the clay solution should be heated to the boiling point to drive out the adsorbed air. The test is made after the solution has cooled.

The lower limit of the plastic state is determined as follows: Mix a sample of the clay with water until it becomes very plastic or, if it is too soft in its natural state, let it dry until it reaches the consistency required. Then work the sample into several thin threads (diameter about 3 mm.), put the threads together and work them out to threads again by rolling them with the palm on a smooth, clean sheet of paper. Repeat the process until no more threads can be formed, the material becoming brittle and the threads breaking to pieces while worked. Then the moisture content is determined; this value, expressed in per cent of the weight of the dried sample, was called by Atterberg the *lower limit of the plastic state.*

In order to determine the lower limit of the liquid state, take a flat porcelain cup (Fig. 3), mix a sample of the clay with water until it becomes very soft, make out of the mixture a cake about 4 cm. in diameter and 0.8 cm. thick, cut this cake with a nickel scoop into two equal parts and shake the cup. If the two lower edges of the cut do not flow together, add some water and repeat the test. The moisture content at which the lower rims join along a strip of a height of about 1 mm. is called the *lower limit of the liquid state.* Both limits should be determined at least twice.

The difference between the two limits is called the

coefficient of plasticity. A clay is plastic between the two limits, else it is either liquid or semi-solid. The greater the coefficient of plasticity, the more plastic the clay is supposed to be. For materials without any plasticity (typical quicksands or very fine quartz dust) the coefficient is zero, i.e. the two limits are identical. For equal coefficients of plasticity the limit of plasticity may be low or high, depending on the shape of the grains and the percentage of humus constituents present.

The degree of plasticity of two clays with equal coefficients of plasticity may be equal or different according to whether the specific gravities of their grains are equal or different. In order to establish a common basis for comparison, the author expresses the limiting moisture contents in per cent of the space occupied by the solid matter. Care should be taken to use samples which have not previously been dried, as the very great capillary pressures which develop during the process of shrinkage may crush many of the grains and decrease the average size of grains.

The data mentioned represent the minimum requirement for characterizing a clay. Without containing these data, the description of a clay is practically worthless and the recorded phenomena are merely curiosities.

Summary—From the experimental studies described in this and the preceding article we have derived facts which may be summarized as follows:

The cohesion of clay is due to two factors. One of these is the pressure exerted by the surface tension of the capillary water, a force whose intensity exceeds all the other forces the earthwork engineer has to deal with. It may amount to several hundred atmospheres; it compacts loose, colloidal sediments more thoroughly than can be done by artificial means except in the laboratory by using a high-power testing machine. Swelling of clay is nothing more or less than the purely elastic expansion produced by the elimination of the surface tension of the capillary water. Local evaporation of the capillary water or local flooding of the surface of clay deposits produces secondary stresses the intensity of which is far greater than the weight of the heaviest structures and which were found to be the primary cause of many vast soil displacements, known as *earth slips*.

The second one of the factors mentioned consists in the fact that the properties of the water contained in voids of width less than 0.0001 mm. are no longer identical with those of ordinary water. In such voids, viscosity and surface tension are increased (in inverse proportion to the diameter of the voids), and the water loses its ability to evaporate in contact with the air. Thus the capillary water of the clays is to a certain degree solidified by the influence of the forces exerted by the molecules of the solid matter. Due to this fact the capillary pressure assumes far greater values than it would if the surface tension of the capillary water had its normal value.

Capillary pressure plays the same part in the physics of clays as does intrinsic pressure in the physics of solids. Therefore the elastic properties of the clays are qualitatively identical with those of granular solids (rocks, concrete, etc.).

The minimum requirement for describing a clay consists in presenting the following data: Water content, specific gravity of the solid matter, lower limit of the plastic and the lower limit of the liquid state of the clay.

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[1]"Die Beziehungen zwischen Elastizität und Innendruck," K. Terzaghi, *Sitzungsberichte der Wiener Akademie der Wissenschaften*, 1923.

[2]"Progress report of tests on undisturbed clays," J. W. Griffith, *Proceedings, Am. Soc. C. E.*, March, 1922, pp. 557-579.

[3]*Proceedings*, Am. Soc. C. E., August 1920, Plate XI and XII; January, 1922, Plate VII.

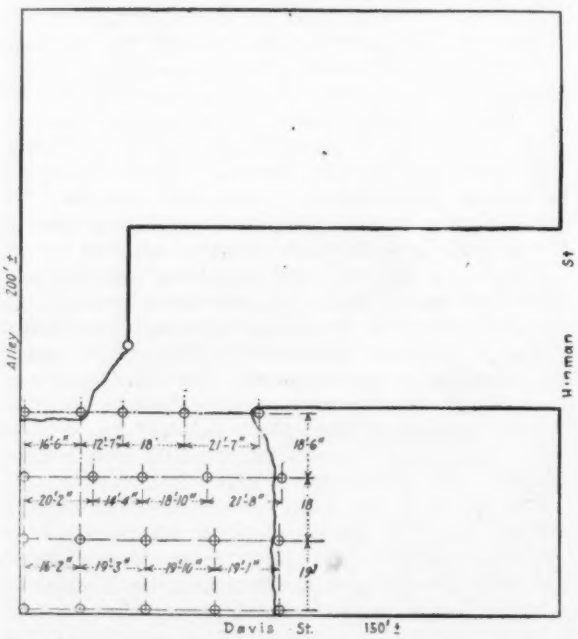
[4]"Die Plastizität der Tone," A. Atterberg, *Internationale Mitteilungen für Bodenkunde*, 1911. Atterberg's method of measuring plasticity was severely criticized by W. E. Emley five years ago (Technologic Paper 169, U. S. Bureau of Standards, "Measurement of Plasticity of Mortars and Plasters"). The arguments advanced by Mr. Emley are justified so far as the plasticity of mortars and plasters for industrial uses is concerned. But they do not seem to affect the value of the method for purposes of soil study.

Concrete Hotel Building Collapses After Cold Weather

Twelve Bays of Four-Story Structure Fall When Forms Are Stripped—Tarpaulins and Salamanders Absent

COLLAPSE of twelve bays of four floors and roof of a reinforced-concrete apartment hotel, the John Evans, occurred at Evanston, Ill., at 2:30 p.m. Nov. 3 while workmen were removing the forms supporting the first floor. An unusual noise was heard overhead and the workmen got out uninjured two or three minutes before a section, 55 x 75 ft. in plan, at the southwest corner of the U-shaped building crashed into the basement. The work had been progressing rapidly and, while apparently no definite record was available as to the time the concrete in the first floor was deposited, superintendents and officials in the owner's organization (the owner was the contractor) estimated that the floor had been in place from 20 to 28 days.

The temperature average, as noted in the accompany-

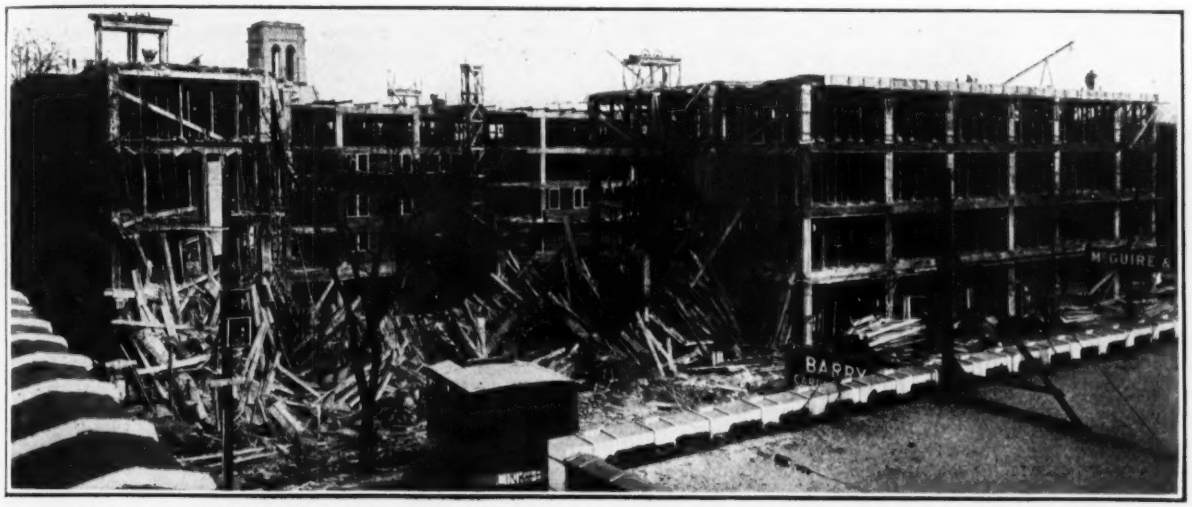


COLLAPSED PORTION OF U-SHAPED STRUCTURE
JOHN EVANS HOTEL COLLAPSE

ing table, for the 25 days previous to the collapse was 41.1 deg. F. which according to the investigations carried out a few years ago at the University of Illinois, would produce concrete having but 70 per cent of the normal strength attained at 70 deg. for 28 days. While the owner states that concrete was protected in the upper floors by canvas and with salamanders for a few days after each section was placed, no tarpaulins show in the photographs taken immediately after the wreck.

This building occupies the northwest corner of Davis and Hinman Avenues and was designed by J. Scanlan, structural engineer, employed by the owner and contractor, Victor Carlson of Evanston. Mr. Carlson's organization has designed, financed, built and inspected a number of buildings including two seven-story hotels and an eight-story office building.

The ground floor is of the flat-slab type with four-way reinforcement, 8½ in. thick, designed for a 100-lb. live-load. The three floors and roof slab are of the "tin pan" concrete-joist type, having 2 in. of concrete over the pans, and with straight rodded columns. These floors were designed for a 40-lb. live-load and would weigh with the centering about 60 lb. per sq.ft. producing 240 lb. per sq.ft. on the first floor if the floors could not support themselves. This would produce working stresses considerably in excess of 600 lb. and probably in excess of 1,000 lb. With the wet mix observed as being used on other parts of the job two days after the collapse, the concrete under the temperature conditions would have considerably less than 1,000 lb. available crushing strength.



CONCRETE OF COLLAPSED TWELVE BAYS OF EVANS HOTEL BUILDING BEING REMOVED BY DRAGLINE
Characteristic hair-like appearance of clean reinforcing rods shows at left side of picture

No records of temperature within the structure apparently were taken.

Temperature Record at Coast Guard Station Four Blocks Away Taken Every 4 Hours for 25 Days Previous to Collapse

Date Oct.	4 A.M.	8 A.M.	Noon	4 P.M.	8 P.M.	Midnight
10	**28**	32	44	44	44	42
11	**41**	43	52	50	48	48
12	**50**	51	56	56	50	50
13	**52**	54	60	56	55	56
14	54	56	56	57	**48**	48
15	**48**	54	50	55	48	48
16	46	48	52	48	46	**45**
17	42	40	42	48	41	**40**
18	40	43	52	52	40	**34**
19	**32**	36	39	39	42	36
20	37	36	46	44	37	**34**
21	**30**	36	36	42	38	38
22	**31**	35	46	46	40	38
23	**42**	43	53	52	46	46
24	46	46	44	44	42	**41**
25	**35**	37	46	42	38	44
26	44	40	42	39	34	**31**
27	**28**	33	33	34	33	36
28	**18**	25	30	34	26	24
29	**14**	16	30	28	25	22
30	**22**	28	32	34	32	28
31	28	32	42	43	36	**34**
Nov.						
1	**33**	46	48	40	38	36
2	**34**	34	**46**	46	45	44
3	**42**	44	47	47	46	44
Average	36.7	39.5	45.0	44.8	41.1	39.5
Average for period	41.1					

NOTE.—Temperatures indicated in bold type are low temperatures for the day.

As to materials, it was stated that sand from the excavation had been used for all of the structure except that portion which had gone down. This was mixed in the proportion 1:2:4. However, for the portion that failed the owner states that sand was obtained from a prominent Chicago building supply dealer and that it was representative of the building sand used in the Chicago district. An examination of the sand from the excavation shows from 8 to 10 per cent clay and silt but, under colorimetric test, although somewhat yellow, is not so dark as to make it unsatisfactory on that score. No test cylinders or checks on the actual strength which the concrete might develop were taken.

The concrete from superficial examination is not very good. It shows the characteristic stripping of rods after failure, but examination of other collapses has shown this is not always a sign of weak concrete.

The owner feels that the failure was due entirely to too early removal of forms. He is planning, however, a thorough investigation by an engineer to establish all contributory factors. The Evanston building commissioner is making a similar investigation.

Discuss City Problems at Middle West Convention

Chief Topics at Des Moines Meeting of American Society for Municipal Improvements Oct. 19-22, Are Paving, Sewage Treatment, Sewerage, Refuse Collection, City Planning and Contract Forms

Separate Sludge Digestion Experience

By L. R. Howson

Of Alvord, Burdick & Howson, Consulting Engineers, Chicago

CONCLUSIONS as to the value of separate digestion of sewage sludge from observations of the construction and operation of plants at Madison and Hartford, Wis., Lincoln, Neb., and Great Lakes, Ill., may be summarized as follows:

Separate sludge digestion tanks have demonstrated their ability to accomplish efficient clarification and sludge reduction and to produce a good sludge which can be readily dried.

Adequate sludge digestion capacity is needed with any type of tank. The separate sludge digestion type of tank, by having its entire sludge capacity available through the withdrawal of the liquor lying between the sludge and scum layers, can probably be built with somewhat less total sludge digestion capacity than tanks of the two-story type where the withdrawal of the liquor is not practicable.

In difficult excavation, such as quicksand or rock, tanks of the separate sludge-digestion type can be built at less cost than those of the two-story type. The construction of shallow settling tanks and the pumping of the sludge *only* to a deep sludge-digestion tank located with its flow level at a higher elevation may result either in the saving of great construction costs, or obviate the necessity of pumping the *entire* sewage flow as would be necessary with a two-story tank.

The separate sludge digestion type of tank is flexible in that the first construction does not limit the relation between settling and sludge storage capacity. Either may be enlarged without the expense incident to enlarging both. It would seem further that separate sludge digestion probably offers an economical means of caring for part of the sludge in some plants of the two-story type in which the sludge capacity has been found deficient, but the flow capacity adequate.

It is possible that the digestion of sludge in separate sludge digestion plants may be speeded up by applying heat artificially, and the sludge storage capacity correspondingly reduced. This procedure has been suggested for experimental observation at the Lincoln plant.

The efficiency of separate sludge digestion tank plants, as of all others, depends as much upon the operator as the designer. Plants of this type may require even more intelligent operation than plants of the two-story type.

With sludge-digestion tanks of the separate type it is possible to mix the sludge by pumping as at Madison or by mechanical agitators as at Hartford, and thus facilitate uniform and more complete digestion.

In the writer's opinion the difficulties experienced with separate sludge digestion have largely arisen from the intermittency and incompleteness of sludge removal from the settling to the digestion tanks. It is believed that the installation of mechanical means to accomplish this more readily and more positively will become more prevalent in the future.

* * *

Progress of City Planning in Rochester, N. Y.

By Edwin A. Fisher

Consulting Engineer to City of Rochester, N. Y.

IN OPERATION for eight years, the City Planning Bureau of Rochester, N. Y., is a continuous body and a part of the executive department of the city. Some of the larger projects carried out are as follows: (1) Acquisition of the abandoned Erie Canal lands for 13 miles through the city at a cost of $1,500,000; (2) construction of an 8½-mile rapid transit and industrial railway on this land, the work is under contract; (3) a link in the comprehensive street plan connecting University Ave. with Andrews St., giving traffic relief; (4) a new Genesee River bridge soon to be constructed to connect a populous section with the business district; (5) acquisition of much property for a wide boulevard and a portion completed; (6) acquisition of property for widening South Ave.

* * *

What Des Moines Has Accomplished on its City Plan

By C. A. Taubert

Acting Secretary, Zoning Commission, Des Moines, Iowa

ACCOMPLISHMENTS since 1916 in Des Moines on its city plan include widening of Grand Ave. from the Des Moines River to Eighteenth St. and from 66 ft. to 80 ft.; widening University Ave. from west Sixth to east Ninth St.; opening and deeding to the city a 4-mile diagonal street to the northeast city limits, by Frederick M. Hubbell; opening a thoroughfare in a ravine to give outlet to the northwest corner of the business district at a cost of $900,000; unification of the east and west sides by construction and development of a civic center on the river banks and connecting by four bridges.

In 1924 there was started a comprehensive city plan to be used as a basis for zoning. Council will soon act on the zoning plan which is now before that body. The city has developed in the shape of a cross due to absence of diagonal streets with the business district at the geographical center. Most of the streets are less than 66 ft. wide and those that are wider lead nowhere. Of forty-eight arteries to the business section only six reach the city limit. Major thoroughfares to carry the bulk of the traffic on existing streets as a basis have been designated and it is recommended that they be widened and extended to make through routes, particularly anticipating these needs in the undeveloped outskirts. Des Moines has 1,100 subdivisions platted without control but a bill is now before the state legislature to give control.

Only a preliminary railroad report has been made. The situation is far from satisfactory. Too much railroad business is conducted on congested downtown areas where its roads are so hemmed in as to make expansion impossible. However, the general location is fortunate in that the railroads are on the least expensive land and cause a minimum interference with the city's growth. Joint freight stations, gradual elimination of grade crossings and eventually a union station which if not built by public contribution will be a building for combined business and station purposes are other developments.

* * *

How St. Louis City Plan Has Worked Out

By Harland Bartholomew

Engineer, City Plan Commission, St. Louis, Mo.

THE effectiveness of a city plan can be measured by the extent to which it directs all physical growth into channels of orderliness, economy and beauty. In considering frankly the extent to which the St. Louis city plan controls or fails to control all development, there are numerous difficulties encountered that are not uncommon in other cities. One of the greatest of these is the fact that a great majority of new growth is taking place outside of the city limits over which no control whatsoever is being exercised. In one year over 500,000 front feet of property were subdivided in St. Louis County in which not one acre of park was dedicated, nor any street platted in excess of 60 ft. in width. Here was virtually a city of 62,500 people without park or adequate street system. This is due in part to the peculiar political condition resulting from the separation of

St. Louis and St. Louis County at the time of the preparation and adoption of the present constitution of the state of Missouri in 1875. Largely as a result of the appreciation of city planning problems an amendment to the state constitution has been adopted and a joint board of freeholders is now sitting to consider and prepare plans for readjustment of boundaries and possibly the creation of a metropolitan organization adequately empowered to control growth throughout the metropolitan district.

* * *

Causes of Concrete Pavement Failures

By H. Eltinge Breed
Consulting Highway Engineer, New York City

COLD weather work has been followed by a string of bad results. There is no gain in strength when the air temperature is around 40 deg. F., there is slight gain at 50 deg., while real strength comes at around 70 deg. Concrete laid at 40 deg. or below will not gain strength until the temperature rises, so if it goes over winter and traffic is allowed on it before the gain comes, the concrete will become shattered. Better concrete can be secured in summer with 7 days curing than can be had in November with 21 days curing.

Another source of trouble is the use of too much water in the concrete. Concrete around switch points where it is subject to freezing and thawing from the use of salt by the traction companies, shows a marked deterioration over the rest of the pavement. Vitrified brick, granite sets or granite block around the switch points will give better life than concrete.

Another cause of poor results is the use of dirty and unsuitable aggregates. It is dangerous practice to try to use a sand that is under 100 per cent of the strength of Ottawa sand unless one is designing the mix on the strength basis, fineness modulus, water ratio, etc., and has enough control to know that any change in aggregates is taken care of by redesigning the mix. Coated stone or gravel is another source of trouble.

Poor joint construction or no joints at all is one of the greatest troubles. When no expansion joints are provided concrete either expands laterally up to its elastic limit or ruptures along its weakest parts. The rupture may not appear the first year but frequently shows the third or fourth. After some experiences of this kind, which showed first in blow-ups, construction was changed to allow more expansion, and an extra joint 1½ in. wide was provided every 750 ft.

Other sources of trouble with joints are: (1) The submerged joint which on compression spalls out on top; (2) the joint which is not perpendicular to the pavement surface and which, when pressure comes, rides above the adjoining slab; (3) the dowel joint which, having no free end, causes the dowel to buckle and spall out the concrete or raise one slab above the other; and (4) the failure to carry a joint entirely through the concrete which on account of concentrated pressure, shatters the concrete.

And after everything else is as good as it can be, the pavement may be spoiled by over finishing, which draws the poor or fine parts of the sand to the surface and either brings out check cracks or causes the top surface to spall off.

* * *

Thinner Brick Pavements More Used

By Webster L. Benham
Consulting Engineer, Kansas City, Mo.

A VITRIFIED paving brick wearing surface of 3, 3½, or even 4 in. is in many cases unnecessarily thick, heavy and needlessly expensive under present-day traffic, particularly in residential and light traffic streets and roads, and especially where proper attention has been paid to other vital features of pavement design. Practice and experience have demonstrated that plain, wire-cut vertical fibre, lugless brick is the most all-around desirable and satisfactory type for the present-day construction. The 2½-in. brick are of this type. Because of their smaller size and lighter weight, they are more easily handled, not only in the manufacturing operations but in the actual laying of them in the pavement.

The rapid increase in the use of thinner brick can be measured by the fact that, whereas only 6,862,565 of the 2½-in. brick size were used in 1920, there was a total of 17,542,786 brick of 2½ and 2¼-in. size laid in 1924.

During the last twelve years several million yards of 2½-in. vertical fibre plain wire-cut brick have been laid in Kansas, Oklahoma, Arkansas, Louisiana, Texas, Mississippi and other southern and western states. Asphalt filler is used, and the artificial base, where it is necessary, has ranged from 4 to 5 in. in depth. A recent survey showed all of these pavements in satisfactory condition, with the exception of two streets in Oklahoma City where the artificial base gave way and allowed the brick to settle. The use of the thinner section brick in the states above mentioned would indicate that this type of construction is giving satisfaction even in the oil fields where traffic is heavy.

* * *

Syracuse Road Resurfacing Test

By H. Eltinge Breed
Consulting Highway Engineer, New York City

THE Syracuse test road, formally known as Syracuse-Cicero State Highway, was built in 1914 of 1:1½:3 concrete, 16 ft. wide, 4¾ in. thick on the outside edges and 6¾ in. thick at the center. This pavement was still giving good service, but some of the areas were badly cracked. The idea in resurfacing at this time was to utilize all the pavement which still had a salvage value. The old pavement is an example of some that were built with an excessively fine sand of low strength. A traffic count shows 10,000 vehicles over it in 24 hours. The resurfacing has consisted of placing a top course with sections 450 ft. long of the following thicknesses: 2¾, 3½, 4 and 4½ in. Reinforcement of three different weights was used with each thickness. In addition to the resurfacing, a new strip 8 ft. wide, was placed longitudinally to give a pavement 25 ft. in width. The whole job is over a mile in length. The 8-ft. slab was laid in thicknesses of 6, 7 and 8 in. and reinforced with various weights of reinforcing, some of it placed both top and bottom. There were also some unreinforced sections of the 7- and 8-in. thickness. Steel fabric and bar mats were the type of reinforcing used, but there were no combinations of the two. Edge strength on the resurfacing and new slabs was secured in the steel fabric mats by using heavier wire at the edge, and also with sheets of metal wider than the slab, turned down along the edge so that a heavy member is placed at the bottom. Additional end strength for the new slab was secured by cutting a sheet of metal in two and using one-half at each end of the slab with the heavy members running across the pavement. This method means using only one kind of steel on the job. It is an excellent method of reinforcing for loading at unsupported corners, for as the bending moment increases, the area of concrete and steel increases thereby increasing the resisting moment. Bar mats were also used, both single and double layer in part of this work. These were assembled on the ground.

The forms used were of two 2-in. strips of wood bolted together so as to break joints and of the height required for the section for which they were to be used. Iron straps were fitted to the sides and they were placed and held by iron pins set in holes drilled in the concrete. The proper position was secured by the use of blocks around the pins, while top elevation was secured by shims.

Corrugated paper joint material composed of two layers of corrugated paper with a hard paper core was used. Later the core was pulled when the two sides could easily be lifted. After the concrete had hardened, the joint was poured with a bituminous mastic. The paper joint had a metal cap on the upper side to strengthen it.

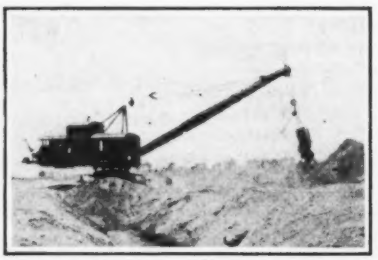
Some of the problems it is hoped to solve when the tests are made on this road after next spring's thaws, are whether the cracks in the old pavement will come through and cause disintegration of the new surface. By how much greater depth of new surface will weaknesses in the old pavement be corrected? What is the minimum thickness on which the new surface can be designed? How much strength is gained through different quantities of reinforcement? How much steel is it economical to use?

Imperial Valley Drainage Problems—Progress in Solution

System of Open Drains, of Which 131 Miles Is Already Built at Cost of $12,427 per Mile, Will Lower Water Table—Topography and Soil Conditions—Methods of Construction and Maintenance

By D. W. Murphy
Consulting Engineer, Los Angeles, Calif.

IMPERIAL VALLEY, broadly speaking, lies partly in California and partly in Mexico (Lower California). The portion within the United States, with which this article deals, is the most southerly of the interior valleys of California. It was once the northern portion of the Gulf of California, filled by sedimentary deposits brought down by the Colorado River. It may be described as the northern slope of the delta which was carried across the ancient gulf. The ridge or backbone of this delta, trending roughly northeast to southwest, has an elevation of about 120 ft. above sea level at its head, or upper end on the river, and about 30 ft. at its lower end, where it joins the higher lands on the west. The irrigated area in the United States ranges from about 30 ft. above sea level at the international boundary to about 235 ft. *below* sea level near the margin of Salton Sea in the northwestern portion.



FIGS. 1-3—DRAINAGE WORK IN IMPERIAL VALLEY, CALIFORNIA

Fig. 1—New River channel west of Brawley, about one-quarter mile wide between top of banks, and 50 ft. deep. This channel serves as one of the two main drainage outlets.

Fig. 2—Dragline excavation of a drainage channel. Fig. 3—Flume carrying an irrigation canal across a drainage channel.

Character of Subsoil—Some of the valley material was laid down under water while the area was still gulf, some was deposited in the beds of streams which cut their way across the delta during its formation, and some was brought in by thin sheets of water which spread over the land or filled shallow depressions. The result is a valley fill which varies from medium sand, carried in the stream beds, to very fine and nearly impervious silts which were laid down on flat slopes or depressed areas. The material is generally stratified; but the strata are so broken that they are not continuous over large areas. Vertically, moreover, variations occur often within shallow depths, so that an area of sand is generally underlaid at a depth of a few feet by fine silts, and vice versa. In some areas various fine silts have been encountered to a depth of 30 ft. or more.

The soils may be classed as tight or relatively impervious, due to the preponderance of fine materials in them. In some places, especially along old beach lines, where sand strata are more common, the soils are porous and permit lateral movement of ground water.

Little definite information is available concerning the depths to ground water over the valley prior to the beginning of irrigation. From the records of a few wells which were put down, it is learned that water was found at depths varying from about 50 ft., as a maximum, to 10 ft. or less. The greatest depths were found in the higher portions, near the international boundary, the shallowest along the eastern side of the valley.

From the best information available, it seems probable that the valley fill was saturated up to within 50 ft. of the surface, or less, prior to irrigation. Along the eastern side of the valley, the elevation of ground water is probably influenced by runoff from the mountain area to the north and east which finds its way into porous subsurface strata. This runoff, which occurs at rather infrequent intervals, is in part absorbed by the stretch of sand hills and adjacent mesa which lie between the valley and the mountains. The rainfall in the valley area is so small (an average of 3.2 in. per annum for the past 20-year period) that it has little or no effect on ground water conditions.

In dealing with the drainage of the valley lands, it is necessary generally to consider only excess waters which find their way into the soil through canal losses and excess irrigation.

Irrigation—Since 1902 the area irrigated has increased from nothing up to nearly 400,000 acres. During 1924 there were delivered to the lands in the United States 1,050,700 acre-ft. of water, or 2.92 acre-ft. per acre of the 359,300 acres actually cropped. It is not possible to determine the actual distribution of water to these lands. The length of irrigation period during the year varied greatly for different areas and crops. Over some areas water was applied practically throughout the year. The amount of water used for different crops also varied greatly. It may be reasonable to suppose that the amount applied annually to some areas is 50 to 100 per cent more than the valley average.

Rise of Ground Water—After irrigation had been in progress for a number of years, the water table on various small areas was observed near enough to the surface to bring up and concentrate harmful alkali salts on the surface through capillary action and evaporation. The lands also soon became unfit for cultivation on account of their seeped and waterlogged condition. Gradually the affected areas grew in size, until in 1921 conditions became serious enough to call for remedial measures. These were necessary not alone to restore to cultivation lands that had been rendered useless, but also to prevent the further spread of seepage.

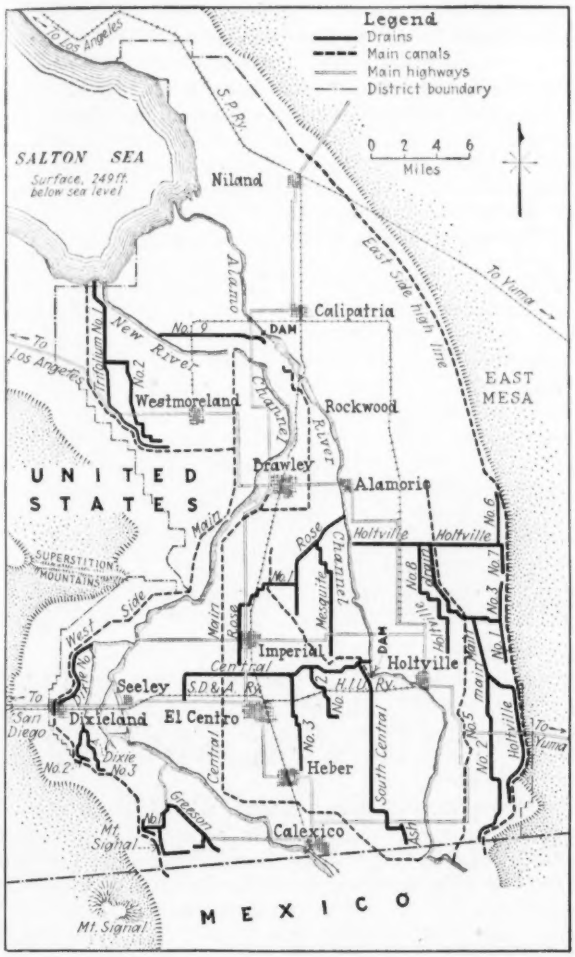


FIG. 4—IMPERIAL VALLEY DRAINS, COMPLETED AND UNDER CONSTRUCTION

At that time, it was estimated that at least 40,000 acres of lands formerly cropped were unfit for cultivation. Over a much larger area the water table was high enough to be regarded as a source of danger to the lands. On account of the fine texture of the soil and resultant high capillary action, surface concentration of alkali and consequent deterioration of land was rapid when the water table once rose near to the surface of the soil. Cultivation also soon became impossible due to the excess water in the soil.

Problems Involved in Drainage—Preliminary investigations made it evident that leakage from canals and underground wastes from irrigation supplied the excess ground water. Some lands adjacent to canals were badly affected; others similarly located were not. Seeped lands were also found on areas so located that canals and lateral losses did not reach them. In these areas, the rises in ground water were due to excess irrigation.

On account of the generally impervious nature of the soils the lateral movement of ground water is slow. The elevation and slopes of the water table, consequently, vary greatly in different parts of the valley. Small isolated spots of badly seeped land are frequently found adjacent to or surrounded by areas of low water table. The waters lost from canals and laterals move slowly through the soils, so that ordinarily only a narrow strip of land adjacent to the canals or laterals is seriously affected by leakage from them. On the eastern side of the valley, where the old beach sands are more prominent, the movement of ground water is greater, and losses from canals have had a greater effect in causing damage there than generally in the valley. It was in this area that seepage first began to do serious damage.

FIG. 5—SECTION OF DRAIN SHOWING DISPOSAL OF SPOIL

It became evident that the plan of drainage must provide for taking care of canal and lateral losses and also the irrigation waste, which slowly percolates downward and causes a general rise of the water table. To accomplish this, it is necessary to provide drains in some areas for intercepting waters moving down slopes, and in others to relieve more or less flat areas under which there is a high water table.

Natural Drainage Conditions—The drainage of the valley is into the Salton Sea, which is the lowest portion of the valley itself. The only outlet from this sea is evaporation. At the present time, waste from canals and surface waste from irrigation are the largest sources contributing water to the sea. The amount of drainage water is gradually being increased as additional drains are constructed. Surface runoff from the watershed area surrounding the sea is practically negligible.

There are two natural waterways: New and Alamo River channels, which cross the irrigable area and empty into the Salton Sea. These channels were first eroded by the Colorado River traversing the delta and discharging into the Salton Sea during prehistoric times. They were deepened and enlarged during the period 1905-1907, when the Colorado River again found its way into the valley. These channels range from about 25 to 75 ft. deep across the upper and main part of the irrigable area. They serve to carry drainage water as well as canal and irrigation wastes. They are available as outlets for main drains for nearly the entire

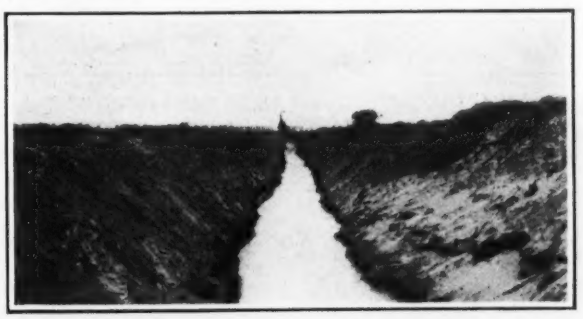


FIG. 6—COMPLETED DRAINAGE CHANNEL

irrigable portion of the valley, and also serve directly as drains for a limited area immediately adjacent to them.

Plan of Drainage Works—In determining the drainage works needed, a general system of main drains was given first consideration. These discharge into the New and Alamo channels or directly into the Salton Sea, and it was desired that, while eventually serving as outlets for extensive lateral systems, they should serve also as

intercepting and relief drains for as large areas as possible.

The open type of drain is necessary on account of the large tributary areas and therefore large flow. In most cases, also, slopes are too flat to make any but open drains feasible. The main drains are being designed and constructed with bottom widths varying from 6 to 12 ft., 1½:1 side slopes and a minimum depth of about 10 ft. The excavated material is used to build a continuous roadway 18 ft. wide and 2 to 3 ft. high on one side of the drain, to be used both as a road for maintenance purposes and as an embankment to prevent excess irrigation water finding its way directly into drains. Excavated material not needed for this roadway embankment is deposited on the opposite side of the drain in a nearly continuous spoil bank.

Work Accomplished—Up to July 1, 131 miles of drain have been constructed. This work involved the excavation of 7,050,000 cu.yd. of earth, and the building of 358 structures (bridges, culverts, flumes and siphons necessary for carrying highways, railroads and irrigation canals over drains).

Excavation has been done generally by means of gasoline-driven dragline excavators of 1½ cu.yd. capacity. The total expenditure has been in round figures $1,628,000, or approximately $12,427 per mile of drain or $0.141 per cu.yd., including depreciation on machinery.

One condition which has in a measure retarded progress and caused a slight increase in the cost of the work is the failure of drain banks to stand and permit the entire section of drain to be taken out at one time.

FIG. 7—DRAIN FILLED WITH TULE GROWTH

Over a considerable portion of the work the water table was above the grade of the drains, at some points ground water was encountered at or near the surface. It was impossible ordinarily to excavate more than two or three feet below the water table without causing banks to cave; attempts to excavate a drain to full section at one cutting (or in some cases even two) were unsuccessful. The excavation was then taken out in successive stages, allowing the bank to drain and become stable between cuttings. The first excavation consisted of a narrow cut along one side of the drain, carried as deep as banks would stand. This was followed after several days (or as soon as the banks had become stable) by a second cutting, which widened and deepened the one already made, and this by others until the required depth and width had been obtained. Ordinarily three cuttings were sufficient, but in a few instances, under the worst soil and water conditions, as many as five were necessary.

Action of Drains—The amount of water collected and discharged by drains has been steadily increasing as construction progressed. The Chief Engineer's annual report of the district for 1924 gives the average quantity of water discharged by drains throughout the year as 45 sec.-ft., exclusive of waste water direct from canals. This is equivalent to approximately 0.53 sec-ft. per mile for 85 miles, the average length of drains in operation for the year.

The average content of harmful alkali salts in this

FIG. 8—DRAIN CLEANED BY CLAMSHELL

water is estimated from various analyses to be 0.275 per cent, or approximately 3.75 tons per acre-foot. This shows that 122,000 tons of alkali—over 4,000 30-ton carloads—were removed from the soils of the valley during the year.

These figures show in a manner what is being accomplished through drainage. On account of the large amount of field work involved no definite figures on the lowering of ground water are available, but it is estimated that the drains so far constructed have a direct effect upon the lowering and control of ground water from 40,000 to 50,000 acres, and that they will provide outlets for from 150,000 to 175,000 acres. A large part of the area which had been abandoned before drainage work was started has been rendered suitable for cultivation.

Maintenance of Drains—In order for drains to function properly, their original depth, at least, must be kept clear of obstructions. Filling tends to occur through caving or scaling of banks, and sediment and tule growth form obstructions. The irrigation supply is heavily laden with silt. It has seemed necessary in many instances to permit surface waste from irrigation to discharge into drains and thus dispose of one source of excess water which would otherwise find its way into the soil. Provision is made for desilting this water before it is allowed to enter the drain. To accomplish this requires constant care on the part of a maintenance organization. If sediment is allowed to get into drains, the only remedy is frequent cleaning. If it can be kept out of drains, the amount of cleaning required is reduced.

The common tule (*scripus lacustris*) which grows so vigorously on wet areas, will grow and flourish in shallow or still water, but its growth is checked by deep or moving water. This plant will start in drains soon after they are constructed, and soon cover the bottom and lower portion of side slopes, obstructing the waterway to such an extent that a small flow may be held up to a depth of several feet. If the tule growth is not removed the drain will gradually become filled and in a

few years be practically useless. The remedy is frequent cleaning. Tule roots should not be removed from slopes above the water line, however, as they provide support against sloughing and caving of banks.

To maintain a drain in the above manner, it is necessary to cut a narrow and deep waterway in the middle of the bottom. This tends to restrict the flow to a deep and narrow channel, and in a measure prevents the growth of tule. The growth on each side of the channel serves also to protect banks.

The ideal machine for maintaining a drain in the above manner has not yet been constructed. The ordinary dragline excavator is not well suited to the work on account of the necessary motion of the bucket across the channel and consequent destruction of vegetable growth on the side slopes. A dragline which would work in the bottom along the center line might prove better adapted to the work. The most satisfactory machine so far tried is the clamshell. This permits dropping the bucket vertically in the center of the drain, and excavates a trench the width of the cut of the bucket leaving slopes intact. The machine is slow because of the small size of bucket that must be used in drains of medium dimensions, and also due to the time required in placing and loading the bucket. There is room for improvement in machinery for maintenance of drains; and this will no doubt be accomplished as the need for better maintenance becomes more fully recognized.

Summary—Drainage work in the Imperial Valley is fairly begun, and the results so far attained show that it is possible to protect the soils from becoming seeped and alkaline due to high ground water. It is impossible at this time to form a definite conclusion as to the amount of drainage work that ultimately will be required. This will depend in a large measure upon the care used in preventing waste of irrigation water into the soil.

Construction of the works described above has been entirely by district forces. R. S. Carberry, chief engineer of the Imperial Irrigation District, is in executive charge of construction forces, while F. W. Slattery is in charge of field investigations and plans for drainage work. The writer, as consulting engineer, has outlined plans and given general supervision to the work.

Railway Bridge and Building Papers and Reports

Water Treatment and Electric Pumping for Railway Water Stations, Metal and Wood Sash for Buildings, and Minor Repair Methods Discussed by American Railway Bridge and Building Association at Buffalo

Railway Water Supply and Treatment

By R. C. Bardwell
Superintendent of Water Supply, Chesapeake & Ohio Ry.

In railroad service the quality of water supply for the locomotives is closely related to the operating efficiency of the railroad. Of 350 billion gallons of water used annually for steam consumption on American railways about 50 billion gallons, or 15 per cent, are receiving treatment in some form. At an average cost of 4c. per 1,000 gallons for treatment, this represents an annual operating expense of about $2,000,000. Of 16,000 water stations, about 1,200 add chemicals to the water, and the total investment in water softening plants, including both the inexpensive and the complete types, is at least $10,000,000. These plants remove annually about 50,000 net tons of scale-forming solids, which if allowed to enter the boilers would involve an additional expense of about $13,000,000 for locomotive operation and maintenance.

The oldest form of treatment is the addition of boiler compounds or chemicals, such as soda-ash, directly into the boilers or tender tanks. In some cases this has shown good results with waters of moderate hardness, but its disadvantages are largely in the lack of a dependable system for checking the regularity and proportioning of the treatment. The least expensive system is the addition of soda-ash in the roadside tanks to soften the sulphate or hard-scale forming matter. This method is used extensively on some roads, and with careful supervision it can be made to give satisfactory results, but it has the disadvantage of producing acute foaming conditions unless the concentration in the boiler is kept down by frequent blowing off. Experiments with zeolite softening on the west coast show ideal results for laundries and other industries, but the excessive alkalinity and resultant foaming, together with the high cost, have retarded its use for boiler service. In this system, a filter absorbs the scale-forming elements and replaces them with non-scale-forming alkali salts; the filter is regenerated periodically by soaking in a strong brine solution.

The standard and complete method of water softening consists in adding lime and soda-ash to the water at roadside settling tanks, so as not only to soften the water but to remove the sludge and other suspended matter, thus delivering soft and clean water to the boilers. With proper supervision, very satisfactory results can be obtained and the economies usually far exceed the estimate. The treatment may be continuous or intermittent, and there are numerous proportioning devices on the market. In the continuous process, some experimenting is being done with simply running the chemicals and water together, without special mixing or agitation, into the bottom of the sedimentation tank, although experience with this system some years ago was not altogether satisfactory.

A recent publication of the U. S. Geological Survey shows that of 307 city water supplies, 234 were from surface waters and 73 from wells. This ratio probably holds in railroad service. Surface water carrying mud and sediment in stormy seasons shows the need for improvement; at low-water stages, the water may be clear, but the influence of its dissolved minerals is much more severe. Well waters are clear, as a rule, but are frequently hard and require treatment for economical use in locomotive boilers. Study by a committee of the American Railway Engineering Association indicates that 13c. is a conservative figure of the cost due to each pound of incrusting matter entering the boilers; this cost covers only the effect on fuel consumption, boiler and engine repairs and engine time lost in repairs. Large intangible benefits, such as elimination of engine failures on the road and reduction in delays to train movements, usually far outweigh the tangible savings in fuel consumption and boiler repairs.

For the effective handling of this problem it is necessary to have the closest co-operation of the engineering, maintenance, motive power and operating departments. Furthermore, the best mechanical facilities will function only in a perfunctory manner or fail entirely if not supervised by a careful checking system, and for this reason an organization with definitely fixed responsibility is essential to successful results from railway water treatment.

* * *

Electric Pumping at Water Stations

The advantages of an electrically operated water station may be: Low first cost, low maintenance, low cost of operation, safety and cleanliness. For a new water station, the electrically operated unit holds a decided advantage in housing, as a complete unit, including control, requires only about 30 per cent of the space required for other methods. The average cost of pump, motor and control of a large number of stations, using various types of pumps with

capacities varying from 100 to 600 gal. per minute, is about $1,100. This will compare favorably with a steam plant or an oil station when the cost of fuel storage is taken into account. The maintenance of a well installed electrical unit is usually low.

Unless provision is made for different sources of power, which is usually impossible, secondary pumping units should be installed. A number of railways leave the original unit in place, and a standby. Others provide oil engines. Unless the storage of water is sufficient to provide for the maximum demand for at least 24 hours, a secondary pumping unit should be installed and kept ready for service. This will provide for the shutting down of the electrical unit for inspection and repairs.

The cost of operation depends almost entirely upon the cost of power. Attendance is the largest item of saving, and the slight attention needed can be given by local employees, who are given a small increase in salary. For 20 stations the average saving per month was $86, mainly in attendance. Where it is necessary to prevent freezing, a small electric heater, a stove or an oil lamp is required, or with float-switch control the limit of draw-off may be adjusted so that the pump will operate every time water is taken from the tank. One railroad uses a small drain pipe, so that there is a continuous flow of water. While the motor drive is used with every type of pump, it is particularly suited to the direct-connected centrifugal pumping unit.

Conclusions — Where conditions are favorable and electric power can be secured at reasonable cost, electrically operated water station plants are cheaper to install and maintain than steam plants. Although it is generally understood that fuel-oil engines furnish the cheapest method of power pumping, they can be offset in some instances by the decreased cost of attendance of electrical stations. When considering the installation of a pumping station stress should be laid on the necessity of considering each point strictly on its merits. Estimates should be made of the cost of housing, of pumping by various methods and the type of power used which will give the service required at the minimum cost.

* * *

Metal and Wood Sash for Buildings

FROM the standpoint of lighting, for the same size of opening, wood sash will admit less light because of its bulkier lines and type of construction, while steel sash has fewer and thinner supporting members. For ventilation, tophung steel sash forms a natural outlet for smoke and gases and at the same time does not have the limitations of wood sash in that it may have to be closed for rain storms. A type of continuous sash made from copper-bearing steel is so designed that the leg of the bottom member is flush with the surface of the glass, so that the sash when either open or closed readily sheds water and has not the usual pocket where water and dirt accumulate. Steel sash is used in all construction except frame buildings and with wire glass can be used as a fire retardent. In permanent terminal buildings, solid steel sash is used mostly.

Continuous steel sash, which is specially adapted for sawtooth or monitor roofs, includes solid steel sash, hollow metal sash and a combination of these types. Solid steel sash with ventilators are being used extensively for large terminal buildings of fireproof construction. Hollow metal sash is made as double-hung, hinged, stationary, pivoted or double pivoted; also with vertical and horizontal swing. Sawtooth steel skylights have operating sash by which the upper section, hinged at the top, can be opened for ventilation. This ventilating feature does not interfere with the daylighting.

Installing steel sash in connection with structural steel construction has brought about the standardization of head, sill and jamb details. All punching of the structural steel for the attachments should be done when the steel is fabricated.

In reinforced-concrete buildings, the steel sash is placed with the frame entering the form sufficiently to seal it from the weather. With "steelead" metal skylight construction, all exposed surfaces are either lead or glass and thus not affected by weather, gases, fumes, salt air or smoke. Glass is held on by lead wings without putty or packing. No bolts, nuts or clips are exposed and flashings are hard lead. These skylights do not need painting or other attention.

In the average railroad building, wood sash will generally be double-hung in side walls, and probably center-pivoted in the roof. Under unusual ventilating conditions the smoke and gases must pass out of the highest part of the building. Center pivoted sash do not permit easy egress. For wood sash, the frames are generally delivered on the work before the sash. The sash should be glazed before shipping, to save both time and cost.

Conclusions—Wood sash can be used economically where the inside temperature is more or less uniform; also where smoke, gases and moisture are present. In frame, frame and stucco, concrete block and brick buildings where the use of shades, screens and storm sash is necessary, wooden sash allows the easy installation of fastenings. For other types of buildings, steel sash is well adapted for use and has the additional advantage of fireproof construction.

Steel sash in engine houses or power houses should be scraped and painted at least every other year. In other buildings, painting is generally done with the periodical painting. A maximum glass area is provided by the use of steel sash. Painting wooden sash is not required as often as for steel. Since the quality of lumber for sash purposes is diminishing, wooden sash will require a substitute, which will be a new type of metal sash with frame. Shrinkage, which is typical of wooden sash and casings, is eliminated by the use of metal sash.

* * *

Handling Minor Maintenance Jobs

THE viewpoint of the maintenance force should be in the minds of the designer and the construction forces, but this is too frequently overlooked. Coal-hoist machinery is often located in cramped quarters and it is necessary to break out a wall or remove the hoist in order to change out gear wheels. The location of valves and connections in pipe lines is often such that it is with difficulty they can be repaired. The spacing of manholes in sewers and drains is often too great, resulting in added expense in cleaning. Records of construction are a great aid to the maintenance force, particularly so far as obscure parts are concerned, such as foundations, underground pipes, sewers and pipes in buildings. Minor jobs can be divided into those which can be deferred until the regular bridge or carpenter crew on its routine trip arrives, and those which are of an emergency nature and require immediate attention.

The maintenance department conducts periodical inspections from which a schedule or program of procedure of work is made for the guidance of the maintenance force, but frequently this schedule is followed too literally. A crew may leave a job and a few days later a report of some other defect is received, which need not have been necessary if the foreman had been alert. Supervisors should instruct foremen to use initiative and assume some responsibility. In spite of all precautions there will be minor jobs to care for out of schedule order, and their proper handling will depend upon whether or not they are of an emergency nature.

It may be asked: "Why not arrange with local contractors to care for these small jobs?" But contractors ordinarily do not want to do small jobs on a lump sum or unit basis, and there is the trouble of making contracts and payments. Of 28 railroads, 13 make a regular practice of arranging with local contractors to handle minor jobs, and 8 do this occasionally, while the others do not follow this practice. The possibility of having a list of local parties on whom the agent can call to do odd jobs should receive serious thought.

Consideration should be given to use of materials for making repairs, such as plastic cements and paints for patching or covering metal roofs and gutters, cold patch material for repairing platforms and filling cracks in cement walks with asphalt, tar or pitch. Attention should be given to time and labor saving devices. Motor cars and trucks may expedite the getting of men and material to and from jobs. The truck will be specially valuable for quick movements in large terminals. Proper equipment and tools are essential for economical handling of work.

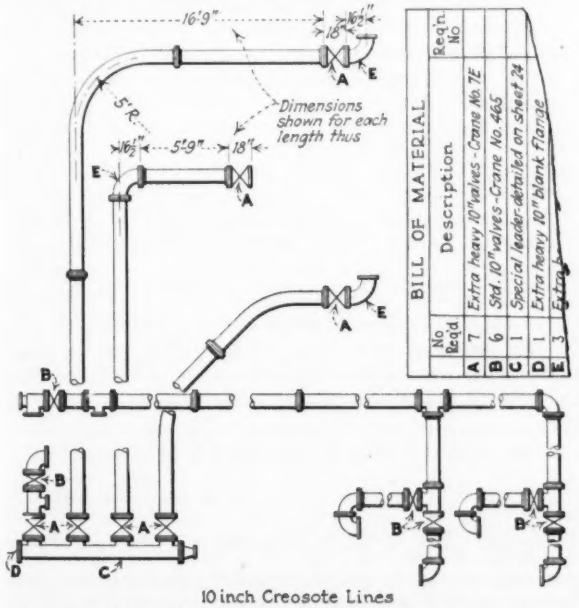
From Job and Office

Hints That Cut Cost and Time For the Contractor and the Engineer

Convenient Scheme for Ordering Pipes and Fittings

IN THE construction of the creosoting plant at the West Oakland (Calif.) yard of the Southern Pacific R.R. some very complex pipe layouts were required in the oil, vapor and water circulating systems. The ordering of pipe for this job and the assembly of the several systems was very much simplified by the use of a method which this company has developed for listing and marking pipes and fittings.

The pipe arrangement is first determined on schematic general layouts. The detail is then lifted from these first drawings and all pipe bends, fittings, etc., are shown in one plane so that all dimensions can be given directly and some letter combination used to designate each piece of pipe and each fitting, as indicated in the accompanying drawing. The letter designations are then arranged alphabetically in a tabulation of the respective items which make up the bill of material. This bill, which is shown on the same sheet that carries the layout sketch, includes all pipe system materials needed even to the gaskets. Specifications on pipes and fittings are also put on the blueprint and thus on this one sheet the factory has complete information pertaining to the particular part of the piping system shown.



SCHEME FOR LAYING OUT AND LISTING PIPE DETAILS

The manufacturer marks each piece, when shipped, with the designation by letter given in the bill of material and also with the sheet number of the blueprint on which the material is listed. The addition of the sheet number prevents confusion, in assembly, between pieces having the same or similar markings and listed on different sheets.

The custom is to design each piping system so that no piece exceeds the 30-ft. limit for convenient shipping length. Joints in sections under 30 ft. long not particularly specified are welded shop joints. All dimensions are given to clearance of faces. Where flanged joints are made necessary by construction requirements but where location is not required to be exact, no dimensions are put on the sheet, thus leaving it to the shop to place such joints where it may be most convenient.

Method Used in Contouring 60 Acres of Rolling Woodland

By S. P. Baird
Engineer and Contractor, Columbus, Ohio

TEN years ago a study of a 60-acre tract of rolling woodland was made in order to ascertain how much money would be required to improve it for residential purposes. The result of this investigation was a map drawn with a scale of 50 ft. to the inch and a contour interval of 2 ft. I have recently run in the streets as finally decided upon. The station numbers on the center line of the streets as run agree within 10 ft. in 2,000 with the line as drawn on the contour map. The quantity of material to be excavated agrees within 3 per cent with the amount found by taking cross-sections from the map. The methods used in making the original map were as follows:

In the first place, a base line was run along one side of the property, measurements being carefully made and points being located at 50-ft. intervals. At as many of these points as possible other lines were run normal to this principal base line; points at 50-ft. intervals were also located on these lines. Accurately measured cross lines were placed as nearly as possible 200 ft. apart. Six other lines were run parallel with the principal base line as a check against the angles of the cross lines.

Up to this point the tract was divided into squares of about 200 ft. with points located on the side at 50-ft. intervals. The sixteen 50-ft. squares inside of each 200-ft. square were then run in by ordinary methods with a plumb line and tape. Stakes were driven at the corners of each 50-ft. square. All stakes in the principal base line were numbered from zero up. The stakes in each of the lines normal to the principal base line were then numbered in the same manner starting at the base line. Thus the second stake up from the principal base line in the fifteenth row would be numbered 2-15. These numbers were painted on the stakes and indicated on the map. Wye level elevations were taken on each stake to the nearest tenth of a foot and in addition twenty-four bench marks were located. The rod on turns was read to 1/100 and all BM elevations were checked by placing them in two sets of levels. All closures were within 5/100 of a foot.

For the contouring procedure, sheets of cross-section paper on which had been plotted each stake with its elevation, were secured along the edges to a galvanized

tube 7 in. in diameter by 22 in. long. The sketcher slipped his arm into the tube and grasped a cross piece at the center by which he was able to turn the tube as required. This method was found to be much more convenient than to use a board.

The party taking contours consisted of a man using a hand level, the sketcher, a rodman and a tape man. The procedure was as follows: The sketcher would call the number of the stake he wanted next and also its elevation. The rod being held at the stake, the level man would place himself so that his eye was on an even foot contour when sighting at the rod. In taking elevations between stakes the rodman soon learned to place the rod very near to the contour line, any correction being indicated by the leveler who always read an even foot. The sketcher kept in good view of the rodman so as to see what angle the contour made with the side of the square. He sketched about 10 ft. each side of the line, the distance being called each time by the tape man. This operation was continued until all the sides of a 50-ft. square were covered. If there was no unusual roughness the contours were run across the square, but if necessary contours on the diagonals were also located.

The total cost of this work including field work, drafting, stakes, paint, paper and tracing cloth was $478 or about $8 per acre. As previously stated, these figures are as of 1915.

Finding the "Critical Depth" in Hydraulic Jump Computations

By Frank S. Bailey

With Metcalf & Eddy, Consulting Engineers, Boston, Mass.

THE accompanying diagram was prepared as an aid in quickly determining the critical depth of water flowing in an open channel, the critical depth being located at the point where the energy head, i.e., the depth plus the velocity head, is a minimum for a given rate of flow.

The diagram is based on an equation given in the article by Julian Hinds, engineer, U. S. Reclamation Service, which appeared in *Engineering News-Record*, Nov. 25, 1920, p. 1034, the equation being $\frac{A^3}{T} = \frac{Q^2}{g}$.

A and T are respectively the area in square feet and top width in linear feet of the water section for the critical depth.

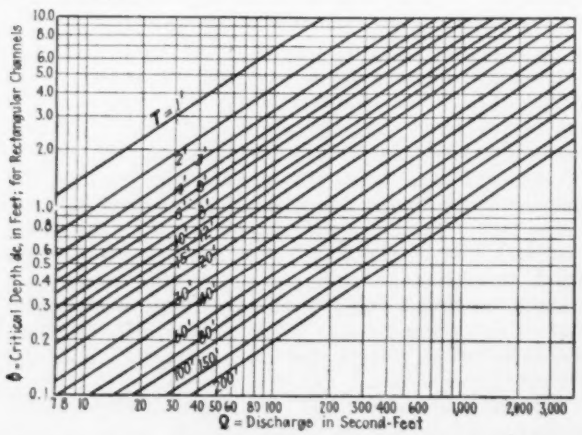


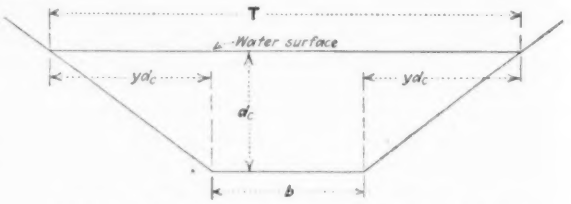
CHART FOR DETERMINATION OF "CRITICAL DEPTH"

From Job and Office

Hints that Cut Cost and Time

Q is the discharge in cubic feet per second and g is the acceleration due to gravity.

In case the channel is rectangular and its width and the discharge are known, the critical depth can be determined at once as the value $\frac{A}{T}$, which is given on the right of the diagram, is also in this case the critical depth.



TYPICAL CHANNEL WITH FLAT BOTTOM AND SLOPING SIDES

For a channel with a flat bottom and sloping sides, as shown in the sketch herewith, the critical depth can best be determined by trial.

If $Q = 100$ sec.-ft., $b = 5$ ft. and $y = 1.5$, we can proceed as follows:

From the sketch, $T = b + 2yd_c$, or $T = 5 + 3d_c$, and $d_c = \frac{T-5}{3}$, $A = (5 + 1.5d_c)\,d_c$

In a rectangular channel with a width of 5 ft. the diagram gives the critical depth as about 2.33 ft., for which the velocity is $\frac{100}{5 \times 2.33} = 8.58$ ft. per sec. In the channel with sloping sides and a bottom width of 5 ft. the sectional area for a given depth will be more than in the rectangular channel and it is probable that the critical depth will be less. We will try a depth of 2 ft.

$$A = [5 + (1.5 \times 2)]\,2;\quad T = 5 + (3 \times 2);$$

$$\text{so } \frac{A}{T} = \frac{16}{11} = 1.45$$

For this value of $\frac{A}{T}$ and $T = 11.0$ the diagram shows the value of Q to be about 110 sec-ft., which is too high.

Next, trying a depth of 1.90 ft., we find that the resulting value of $\frac{A}{T}$ is 1.39 and, as the diagram shows a value for Q of 100 sec.-ft. in this case, it is evident that the approximate critical depth has been found. The velocity is found to be 6.7 ft. per sec.

A similar method may be applied in finding the approximate critical depth for other sections.

In channels which are gradually enlarged, the velocity being thereby reduced, the difference in the velocity heads is sometimes a material factor in determining the slope of the water surface.

In such cases if the depth does not go below the critical depth there is theoretically no loss in the energy head (aside from friction) excepting the loss caused by eddies in cases where the section is not designed especially to prevent their occurrence, as is demonstrated in the article on, "Determining the

From Job and Office

For Contractor and Engineer

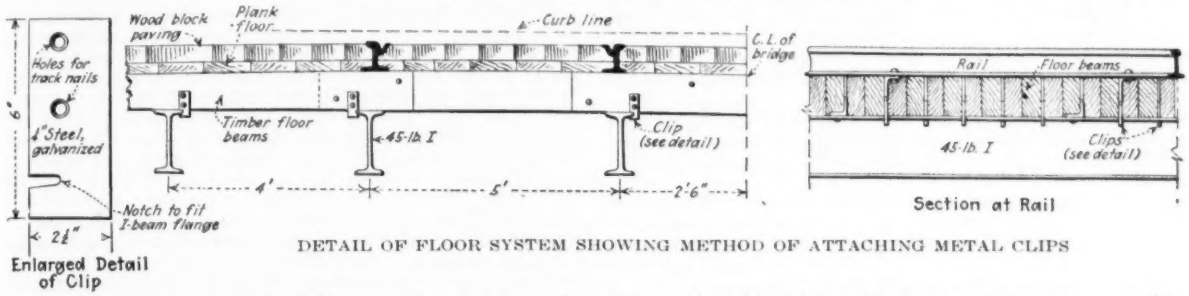
Energy Lost in the Hydraulic Jump," by J. C. Stevens in *Engineering News-Record*, June 4, 1925, p. 928.

The finding of the critical depth is necessary in such cases in order to ascertain whether or not a loss of energy is to be expected.

Metal Clips Fasten Floor Joists to Stringers in Lift Span

IN THE design of the bascule span recently constructed by the city of Seattle over the Lake Washington ship canal on Montlake boulevard, a convenient detail was worked out for fastening the timber floorbeams to the steel I-beam stringers to hold the floor system rigid when the span is raised. This consists of ¼-in. galvanized iron plates cut in 2½x6-in. sizes and notched to fit the flange of the I-beams. These metal plates were nailed to the floor joists by two 3½-in. track nails as shown in the accompanying illustration.

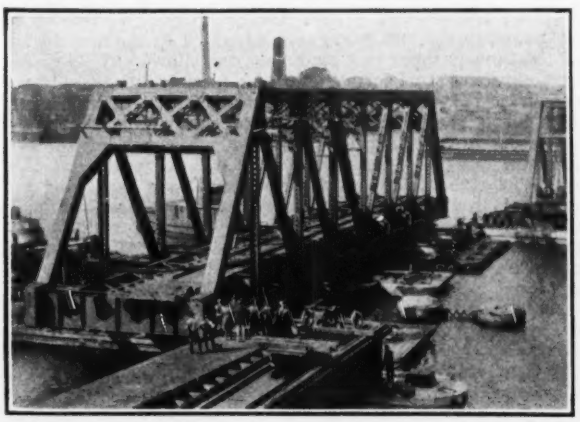
Details of this structure were worked out under the direction of A. W. Munster, acting bridge engineer, city of Seattle.



DETAIL OF FLOOR SYSTEM SHOWING METHOD OF ATTACHING METAL CLIPS

Job and Office Notes

The Florida East Coast Ry., which runs from Jacksonville down the east coast of Florida to Key West, has recently completed the construction of six new bridges, in addition to double-tracking its line from Miami to Jacksonville and building a $3,000,000 terminal near Miami. A total of $85,000,000 is being spent on railway equipment, improvements and developments. To provide for increased freight and passenger travel, large developments are being rushed. The bridge span over the St. Johns River, weighing 1,400,000 lb., was recently set into place in four minutes. The new span is 216 ft. long and carries double tracks. It was erected on specially-driven falsework near the bridge site, when barges filled with water were pulled beneath it. The water was then pumped from the barges by large steam pumps, permitting the barges to lift the span from the falsework. The span was then towed to the bridge site where barges were again filled with water, causing them to lower the span on to bearings on the new concrete piers, rolling into place in four minutes time. Before putting in the new span, the old span was floated out in the same manner. The operation of removing the old span and replacing the new span consumed but six hours with slight interruption to the heavy traffic. The St. Johns River in places is 60 ft. deep; some of the piers of the bridge are 95 ft. below the surface of the water, which gives them 35 ft. penetration into the river bottom. The bridge cost $2,000,000. Five other draw spans were completed the past year by this same railway.



Roadside Water Supplies on four Illinois state highway routes, 4, 10, 24 and 31, have been tagged with safety seals of the State Board of Health. Harry F. Ferguson, chief engineer, states that only 10 per cent of the supplies so far examined have been found safe or free from possible contamination. A water supply, to receive a safe-water seal or certificate, must be so constructed and located that it will be reasonably free from any possible contamination and then analyses must confirm the conclusions drawn from the field inspection as to the probable sanitary quality of the water. No samples are collected for analysis from those supplies which are shown by field inspection to be subject to contamination, thus avoiding needless expense.

At a Paper Mill in Peoria, Ill., which uses a large amount of water pumped from driven wells, several wells became useless owing to the packing of sand against the strainers. To open them back pressure was put on the wells by a fire pump which showed a gage reading of 195 lb. Each well was thus opened and 6 lb. of dynamite was then lowered to the bottom of the well and exploded. The explosion loosened the sand, though it shattered the strainer. A derrick was set up and the loosened sand was removed with a sand bucket fitted with a valve at the bottom. The old casing, which was 8 in. in diameter, was left in place and a 6-in. casing with the strainer was lowered and set in place. The wells now furnish water to the capacity of this casing.

Last Winter the Standpipe at the Michigan State Industrial School at Lansing was found to be leaning after a severe cold spell. Upon investigation by one of the staff of the bureau of engineering of the State Department of Health it was discovered that the tipping was due to water (though the tank had been unused for some time it was full) expanding after it was cooled below the temperature of maximum density when the top was sealed with ice. This pushed out the bottom of the tank and two of the four 1¼-in. anchor bolts were broken off, making the tank lean. The actual displacement from the vertical at the top of the 105-ft. tank was 4 ft. 6 in. The shell at the bottom is ⅜-in. plate and decreases in thickness toward the top. The tank has a diameter of 10 ft. 3 in. To prevent further displacement the superintendent in charge of construction at the school had guy ropes attached to the tank. To bring the tank back to its original position a hole was cut in its side about 6 in. from the base and a flanged coupling was welded around the hole. A steam jet was then screwed into the coupling and steam turned into the tank. Soon after the steam was turned into the tank it began settling back into place.

Letters to the Editor

This department aims to be a forum for the discussion of the views of engineers and contractors. The range of interest should be as wide as possible. Contributors are, therefore, asked to make their letters short.

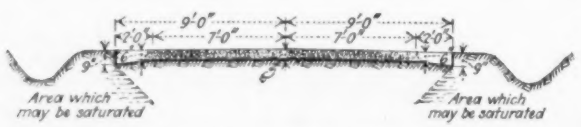
Thickened Edges for Bituminous Pavements

Sir—It seems strange that engineers who are advocates of bituminous base and top pavements have not "in order of better construction and better opportunity to compete" with other types adopted the idea of thicker edge construction, with thinner center. There are many reasons why a roadway 18 ft. wide constructed 4 in. in thickness in the center, for a width of 14 ft., then dropping the subgrade to give a thickness 7 in. at the edges is better construction than building a pavement 5 in. in thickness throughout.

First, the average vehicle is driven with the outer wheel about 1 ft. from the edge of the pavement. By having a thickness of 7 in. at the edge instead of 5 in. there will exist 2 in. of extra dry subgrade, during wet weather, at the point of load.

Second, the heaviest load, on account of crown, falls on the outer wheel where the pavement, if the vehicle is driven 1 ft. from the edge, will be found to be 5½ in. thick instead of 5 in.

Third, on account of the steep rake on the under side of the pavement at the edge, the movement of soft subgrade should be toward the center of the pavement rather than be



CONCRETE PAVEMENT WITH THICKENED EDGES

squeezed outward, where it would have an easy escape at the edge of the pavement. By thus confining the subgrade it will offer greater support to the pavement.

Fourth, to cause a depression in a pavement at any point near the center of the roadway, it will be necessary for the pavement to bend or break at three points, at the bottom of the depression, and at each edge, while if loaded at the edge sufficiently to cause a bend or break, the stress is at one point only. The thick edge provides for this extra stress.

Fifth, an examination of more than 200 miles of bituminous base and top highway has failed to disclose any irregular contour along the central portion of the pavement, while numerous settlements were in evidence along the edges, caused by wet subgrade squeezing outward and upward at the edge of the pavements.

The adjoined sketch shows a design of portland cement concrete pavement quite generally adopted in the western states. On account of the center expansion joint, and the many transverse expansion joints, this type may be broken in the center, as an edge break, or the inner corners may be broken, the inner corners being caused by the center line expansion joint intersection with the transverse expansion joint.

It is my firm belief, after more than ten years' experience and observations throughout California, Oregon, Washington, Utah, Idaho and Montana, that a properly designed mixture of hot mixed bituminous pavement, of 2-in. base and 2-in. top, with 7-in. edge, will render far greater and more satisfactory service than will a portland cement concrete pavement, constructed as shown by the sketch, and I sincerely hope that engineers throughout the country will adopt this design for highway construction.

M. L. GORDON.

Kalispell, Mont.
Oct. 14, 1925.

On Staking Out High Water Lines

Sir—Something more may perhaps be usefully said, in extension of your notice of the New York Attorney General's decision (*Engineering News-Record*, Oct. 29, 1925, p. 731) as to what constitutes a deed "high water line" on the ocean front of the seashore of the state of New York.

Says the surveyor's learned official guide and assistant: "'High water line,' as a title boundary.....means the line of ordinary high water, and not the line of the semi-monthly spring tides. Such line is generally evidenced by the limit to which upland vegetation survives, or would survive except for the presence and action of the waters."

Imagine, if you please, the predicament of the learned attorney general, were he given a sharpened stake of wood or iron, and told to drive it, in accordance with this ruling, into a wide, sand beach, or an improved town lot, at a point where it would mark a lawful high water line; being at a point where upland vegetation would have survived, if there had been any, anywhere in the vicinity, for the past several or many centuries; or where it has been eradicated; or to do this with precision even where there is upland vegetation.

Again: as the height to which "ordinary high water" rises, increases from high water neap tide, to high water spring tide (and again diminishes to high water neap tide), not by a sudden step or jump, as the opinion seems to suppose, but gradually, as influenced mainly by and throughout the revolutions of the moon about the earth, and the conjunctive influence or the reverse, of sun and moon, and according to a most complex curve of increment and decrement; and further, as the locus of these several curves themselves varies throughout the year, not to mention the effects upon it of storms, and of other disturbing causes; how, guided by the instructions conveyed, and no other, is anybody, administrative officer or plain citizen, to stake out such contemplated dividing line, in the absence of surviving or never existent vegetation?

Mean high water for a month, for a year, for many years, can by observation be determined; by observing water heights alone (without taking cognizance of grasses or of flowering plants). But where, according to the code promulgated, is the line to be drawn between water heights to be noted and average, as of the "ordinary" tidal kind, and those coming under the generic name of "spring" tide high water heights? Surely not by excluding only those heights which obtain on the single days of new moon and of full moon, as the opinion quoted, appears to prescribe.

Lest this letter be destructive only, and in no manner constructive, let the proposition be put forward, that, as laymen consult counsel in difficult cases of the law, counsel may with the like profit consult engineers in matters appertaining to the "forces of nature," which it is the province of engineers to observe, define and utilize; and that this be done before the issuance by the inexpert of codes for the action and control of the last named servants of the public.

C.H.

New York,
Nov. 2, 1925.

Steel Forms on Tunnel Curves

Sir—In your issue of Oct. 15, 1925, p. 623, is an article covering the building of the Amaga Railway Extension in Colombia, in the last paragraph of which in the description of the tunnels there is a statement as follows: "It is stated that sectional steel forms cannot be used as the tunnels are not on tangent but on curves of different degrees; otherwise such forms would reduce the cost of concreting."

We desire to take this opportunity of correcting the impression that prevails in the mind of the writer of this article, for the reason that steel forms have been used for the past twelve to fifteen years on all sections of tunnel construction, with all kinds and variations of radii in the same tunnel. You perhaps will recall some of the curves in the New York aqueduct, on which steel forms were used, and this problem has been successfully solved in the construction of railway tunnels and all classes of tunnels, irrespective of the variation of the radii.

BLAW-KNOX COMPANY,
BY HOWARD B. LOXTERMAN,
Assistant General Sales Manager.

Pittsburgh, Pa.
Oct. 28, 1925.

News of the Week

CURRENT EVENTS IN THE CIVIL ENGINEERING AND CONTRACTING FIELDS

Working Stresses Discussed At Two Meetings

An Increase in Stresses for Steel Opposed—Stresses in Wood and Concrete Considered

A special meeting of the Structural Division of the American Society of Civil Engineers was held on Oct. 28 to discuss the subject of working stresses, which at present is exceedingly active. On Nov. 2 the subject was discussed at a joint meeting of the Philadelphia Section of the American Society of Civil Engineers and the Philadelphia Engineers Club.

At both meetings the discussion was based on three recent documents: The proposals of the American Institute of Steel Construction for an 18,000-lb. working stress in structural steel, the report of a committee of the American Society of Civil Engineers on stresses in structural steel for buildings, recommending a basic stress of 20,000, and a recently issued revised draft of working-stress recommendations for building codes by the Building Code Committee of the Department of Commerce, covering wood, steel and concrete.

New York Meeting

Prof. F. E. Turneaure opened the New York meeting with an incisive discussion of the general aspects of the subject, followed by specific comment on working stresses for steel and for concrete. With respect to steel he approved of the general consideration set forth in the report of the A.S.C.E. committee, and declared the old accepted 16,000-lb. stress to be ancient and to have been accorded too much importance. Deductions as to proper practice in buildings cannot be made from bridge practice. Reinforced-concrete, having no hampering traditions, has a different history of working stresses. Building codes, Prof. Turneaure declared, should be balanced in their working-stress clauses, and should be based on average rather than expert ability, but on the other hand should not make allowance for gross errors. Uniformity of code provisions is hardly to be desired, however; "it would probably be a misfortune if one single building code were to be adopted by all governmental units in the country." The condition of actual building practice and code enforcement is also a factor to be considered, and on this point experience in the enforcement of codes in Wisconsin suggests that the quality of field concrete is far from ideal.

The strength requirements set for concrete by the Building Code committee and used as basis for its working stresses are excessive, the shear stresses are dangerously high, and the column stresses are questionable, while the steel stress fixed for reinforced-concrete work is quite conservative. The working stress of 18,000 lb. for structural steel is conservative, more so than the concrete stresses, but probably as high as is proper in a building code, said Prof. Turneaure, although he favors 20,000 for work done under good design and execution. The working stresses specified for steel columns, on the other hand, he criticised as too low.

A. E. Lindau, addressing himself particularly to concrete stresses, emphasized the fact that concrete is free from risk of fatigue in service because its strength increases with age, and the fact that field tests usually show strengths higher than laboratory or design tests. He holds that it is important to consider a revision of working stresses for concrete, and thinks

(*Continued on p. 316*)

Engineering Fifty Years Ago

From *Engineering News*, November, 1875

The Highest Signal in the World

A NEW surveying signal has lately been erected on the summit of Mount Shasta, California, by the Coast Survey Department. The signal is a hollow cylinder of galvanized iron, twelve feet high and two and a half feet in diameter, surrounded by a cone of nickel plated copper, with concave sides, three feet high and three feet in diameter at the base; and its altitude is, according to the observations taken by the members of the Coast Survey, 14,402 feet. The nickel plating of the signal is a brilliant reflector, and will, from 6 to 9 A.M., and from 3 to 7 P.M., reflect the sunlight in such a manner that the reflection can be seen from the valleys and the mountains from which the summit of the mountain is visible. It is believed that it can be used for observations at a distance of one hundred miles, and possibly further.

Prof. Ira O. Baker Dies

Ira Osborn Baker, emeritus professor of civil engineering at the University of Illinois, Urbana, Ill., died Sunday Nov. 8.

An appreciation of Prof. Baker's contribution to civil engineering will be published in next week's issue.

Many New Projects Approved at Recent Election

Bond Issues Passed—City Manager Plans Approved—Other Special Issues

Many special issues of particular interest to civil and municipal engineers and to contractors were voted upon at the November election just passed. The largest single items approved were the two amendments to the New York State constitution providing for bond issues of $300,000,000 for grade crossing elimination and $100,000,000 for state buildings, noted in our issue of last week, p. 769, and the most important from a professional standpoint was the amendment to the New York State constitution which eliminates the elective position of state engineer and makes the state engineer a subordinate position in the Department of Public Works.

San Francisco defeated the proposition to buy the Market St. Railway Co.'s property for $36,500,000 by a vote of about 8 to 1. Municipal distribution of power from the Hetch Hetchy power plant was advanced by the election of the "minority group of supervisors" which is not in favor of the sale of this power to a private corporation for distribution in the city.

Out of a total of $26,610,000 for general city improvements and betterments at Kansas City, Mo., only two items were approved. They are $1,200,000 for extending the city's fire department and $1,200,000 for hospital buildings.

Detroit, Mich., passed a number of charter amendments, one empowering the city council to make improvement districts and to vacate streets and alleys therein and generally adjust the property limits, one empowering the council to fix the lines of streets and designate a future date when improvements so proposed shall be accomplished, one to create a sinking fund for street widening assessments, another for the condemnation of property at the Belle Isle bridge approach, and one looking toward the provision of rapid transit for the city.

Dayton, Ohio, approved bond issues, one for $400,000 for a new children's home and one for $875,000 for the construction of new bridges in Montgomery County. Charles H. Paul, former chief engineer of the Miami Conservancy District, was elected a member of the city commission.

Voters of Toledo, Ohio, rejected all but one of a list of 18 bond issues totaling $32,500,000. The one which carried was an item of $3,000,000 for grade-crossing elimination.

A special road loan election in Wake County, Raleigh, N. C., gave a majority for the $1,300,000 bond issue.

Sandusky, Ohio, voted $60,000 for grade-crossing elimination, and Fos-

toria voted $50,000 for hospital construction and $175,000 for water-works development.

In Union County, Fla., a bond issue of $360,000 for road construction was passed, which, when added to the $440,000 voted on recently, makes a total of $800,000 for hard-surfaced roads in the neighborhood of Lake Butler. District road bonds voted on at Moorehaven, Fla., to the amount of $500,000, were approved. Williston, Fla., voted a total of $150,000 including $65,000 for a sewage disposal plant and $50,000 for extensions to the water-works.

Henry County, McDonough, Ga., approved an issue of $200,000 for road work, its part of an $800,000 project in which the state and federal government join.

Large sums of money were voted by cities and towns in Pennsylvania and equally large amounts were rejected. Bond issues approved include $750,000 for a joint city and county building at Reading, Pa.; $250,000 for public recreational purposes, and $1,250,000 for the completion of a school building program at Johnstown; $125,000 for improvements in Mt. Lebanon township; a total of $3,000,000 by Lancaster and York Counties, for a publicly owned bridge between Wrightsville and Columbia. Swatara township voted $100,000 for a vocational school and $90,000 for a water system; Royersford voted $100,000 for municipal improvements and $75,000 was provided at Doylestown for the same purpose. Huntingdon, Pa., voted $200,000 for the erection of a water filtration plant and reservoir for the borough owned water supply. Johnstown, Pa., passed bond issues of $1,250,000 for schools and $250,000 for a municipal athletic field.

A $2,500,000 bond issue for the construction of a new sewer system at Chester, Pa., was approved as the result of a lively campaign. Some time ago the Pennsylvania state authorities forbid the building of extensions to the present sewer system because it is contaminating the water supply. Towns in Lackawanna County, Pa., voted a total of $340,000 for new schools.

Two counties in New Jersey, Essex and Union, approved bond issues totaling over $1,000,000 for park purposes.

Relocation of the tracks of the Chicago & Northwestern Ry. through the villages of Whitefish Bay and Shorewood in Milwaukee County, Wis., was approved by the electors and $65,000 was voted as their portion of the cost.

The citizens of Bloomington, Ill., voted $700,000 for the establishment and maintenance of the Bloomington and Normal sanitary district, sewage disposal plant, and sewage system.

At Cincinnati, Ohio, bond issues totaling $6,557,000 all met with approval. Most of the new work is to be on streets and roads, the largest single item being $2,718,000 for widening Ninth Street. At Covington, Ohio, there seems to be some doubt as to the result on the $425,000 bond issue for schools.

Cleveland, Ohio, passed bond issues totaling $10,166,000, including over $1,000,000 for school purposes. Akron authorized an issue of $3,500,000 and Canton, Ohio, an issue of $904,000.

School bonds were an active issue at Louisville, Ky., where a two-thirds majority is required. The total issue voted is about $5,000,000. Houston, Texas, also approved of a school bond issue of $4,000,000.

By a majority of 13,292, amendments to the charter of Rochester, N. Y., providing for a city manager, were adopted on Nov. 3, effective Jan. 1, 1928. The plan was outlined in these columns, Oct. 22, p. 692.

A majority of 425 votes against giving the city council of Boulder, Colo., enlarged control of the city manager was recorded.

The 3 to 2 vote of the City Commission of Bayonne, N. J., some weeks ago, to join the North Jersey Water Supply District, was nullified Nov. 3 by a popular vote of 8,874 against 3,615 for the proposal. An impounding dam on the Wanaque River and an aqueduct therefrom to Newark, N. J., with branches to other cities and towns has been under construction for some time past by the North Jersey District Water Supply Commission.

Norcross Tablet Unveiled at Georgia Tech

A memorial tablet to Paul Norcross, one of the 23 victims of the Memphis boat disaster on May 8, was unveiled Oct. 30 on the grounds of the Georgia School of Technology, Atlanta. It was erected by his fellow engineers and the exercises under the auspices of the Georgia Section of the American Society of Civil Engineers, carried out in a downpour of rain, were attended by more than 1,000 persons.

John F. Coleman, representing the American Institute of Consulting Engineers, officiated as the presiding officer. Robert Ridgway, president, American Society of Civil Engineers, made an address on "The Contributions of Paul Norcross to the Engineering Profession." The tablet was presented to the school by George W. Fuller, Chairman of the Engineering Board of Review of the Sanitary District of Chicago. J. E. Gibson, representing the American Water Works Association, unveiled the tablet and Dr. M. L. Brittain accepted the memorial for the trustees of the school.

The memorial consists of a seat carved out of a solid block of granite. On the back of the seat is a bronze tablet with the following inscription:

PAUL HOWES NORCROSS

Born Atlanta, Georgia, January 19, 1880; lost in "Norman" boat disaster, near Memphis, Tennessee, May 8, 1925. Graduate Georgia School of Technology 1902; director American Society of Civil Engineers; trustee elect American Water Works Association; member American Institute of Consulting Engineers; member and secretary the Engineering Board of Review of the Sanitary District of Chicago, 1924-1925.

This tablet was erected by his fellow-engineers as a permanent record of their esteem for him as a man and as an engineer.

Howard Succeeds Felton as Head of Chicago Great Western

Col. Nathaniel L. Howard, general manager of the Chicago Union Station Co., has been elected president of the Chicago Great Western R.R. to succeed Samuel M. Felton who has been elected chairman of the board of directors. Colonel Howard was born in Fairfield, Iowa, in 1884 and graduated from West Point in 1907. He began his railroad work in the engineering department of the Chicago, Burlington & Quincy R.R. but soon entered the operating department. In 1917 he went to France as lieutenant-colonel of the 13th Engineers, later being commissioned colonel. After the war he became assistant federal manager of the C. B. & Q. R.R., and in 1924 was appointed general manager of the Chicago Union Station Company.

Mr. Felton also began his railroad career in the engineering department, starting as a rodman on the Chester Creek R.R. in 1868. During the war he served first as adviser to the chief of engineers, U. S. Army and then as director general of military railways.

Gilboa Dam Slopes Changed

During the past summer it was found that the slopes of the dike of the Gilboa dam, on the addition to New York's water supply, as originally planned were not sufficiently flat to accommodate the material which was available for that construction. These slopes were therefore flattened from an equivalent of about 1 on 3 with berms to 1 on 5 without berms. Since these slopes have been flattened the situation has been stabilized and the movements which were in evidence during the summer and which were the subject of much comment in the daily papers in New York, have ceased.

This dike forms the western abutment of the Gilboa dam, now being constructed across Schoharie creek in Schoharie county, New York. It is curved in plan and is 700 ft. in length. The stream control opening in the Gilboa dam will be closed during the late spring of next year and the Schoharie reservoir will then go into service as the last unit on the Catskill Water Supply System.

Ry. B. & B. Plans for 1926

Following the convention of the American Railway Bridge and Building Association, reported in *Engineering News-Record* of Oct. 29, page 732, subjects for committee reports for the 1926 meeting are announced as follows: (1) Application of paint to steel structures; (2) use of motor cars; (3) roofing materials; (4) protecting overhead structures from locomotive gases; (5) methods of repairing disintegrated concrete; (6) use of compressed-air appliances; (7) pumping equipment for various conditions of railway water service. The 1926 meeting is to be held at Richmond, Va. At the Buffalo meeting, 180 members were present, with a total registration of 355.

Clearing Bureau to Investigate Contracts and Bidders

Articles of incorporation have been taken out in California for "The Clearing Bureau of Contract Investigation," a non-profit making corporation formed by joint action of eight surety companies in co-operation with the Contractors' Association of Northern California and the California Department of Public Works.

The purpose of the new organization is "to improve contracting service, to stabilize the business and procure the best results for the public. It is proposed to do this by gathering and making available for any of the parties interested all information concerning public works contracts, the bidders and those to whom the contracts are awarded." The following summary of the need for such a bureau, how it was organized and the proposed functions is taken from an announcement given out by the new organization on Oct. 30.

"Much criticism has been leveled at contractors, surety companies and public officials for the awarding of contracts to irresponsible contractors. Surety companies have been blamed for bonding them and the business of contracting has been rendered unprofitable and has fallen into disrepute by the actions or defaults of such contractors. Something is needed to weed out the irresponsible contractor and to place the business upon a higher plane. Such is the aim of the bureau and the three interests most concerned in the matter have chosen this method of co-operating to that end."

The bureau has been formed solely and exclusively to gather and disseminate information in the interest of safer underwriting of contract bonds, procuring better bidders and contractors, and better performance of contracts. It will have nothing to do with awarding contracts, furnishing bonds or influencing directly or indirectly the actions of any contractor, surety company or public official. Much of the work of the bureau will relate to engineering information and technical data involved in bidding on and completing contracts. The bureau will select for each investigation the particular engineer whom it may consider best to advise with about the contract under consideration.

All persons interested in public works contracts are welcome and are entitled to service free of charge from the bureau which is supported by voluntary contributions from the principal interests mentioned in the following:

The surety companies represented include the National Surety Co., Fidelity and Deposit Co. of Maryland, United States Fidelity & Guaranty Co., Globe Indemnity Co., Aetna Casualty & Surety Co., Maryland Casualty Co., New Amsterdam Casualty Co., Massachusetts Bonding & Insurance Co.

The three incorporators, selected to represent the public, the contractors and the surety companies are R. M. Morton, California state highway engineer and director of public works; H. E. Wood, superintendent, bonding department, Aetna Casualty & Surety Co., acting on behalf of all of the surety companies represented; and Earle G. Lloyd, manager of the Contractors' Association of Northern California. These three men constitute the board of governors of the bureau. The secretary-treasurer is H. J. Douglas, United Bank & Trust Bldg., San Francisco, formerly manager of the American Surety Co. Upon unanimous approval by the board of governors, membership will be open to surety companies, contractors associations and others interested in the welfare of the contracting business.

1,000 Kilometers of Highway to Be Built in Salvador

Contract for the construction of approximately 1,000 km. of modern highway has been awarded by the Republic of Salvador to R. W. Hebard & Co., Inc., of New York, and Rene Keilhauer. The work is to be financed by special taxes and a percentage of customs collections. The construction program will be spread over a period of five years.

Tunnel Proposed Under River at Detroit, Mich.

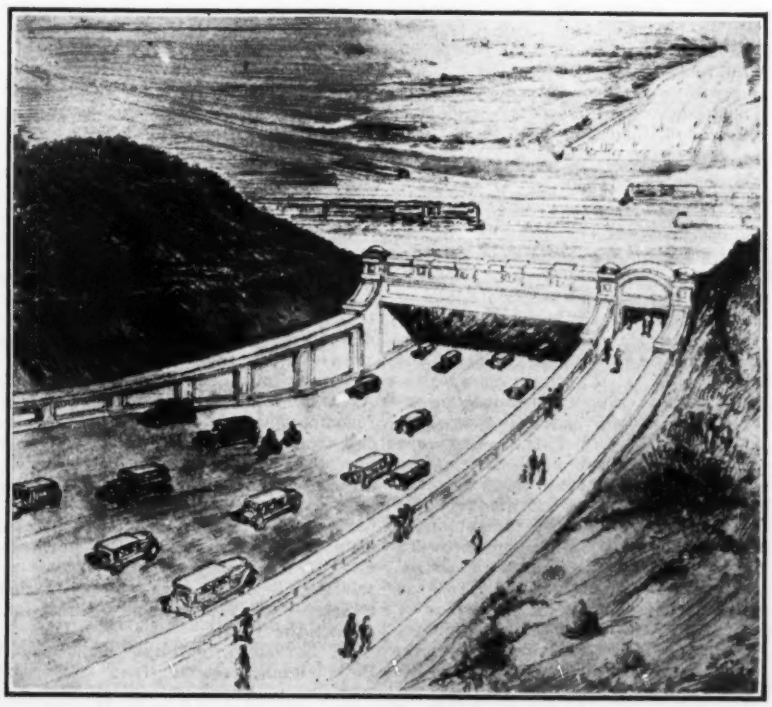
For the purpose of constructing a tunnel under the Detroit river, between Windsor, Ontario, and Detroit, Mich., the Detroit River Subway Company has been granted a federal charter in Canada, with a capital stock of $5,000,000. The company has power to amalgamate or unite with any similar company that may be formed in the state of Michigan with the same object in view.

Underpass Near San Francisco to Cost $260,000

Plans are now being prepared by the bridge department of the California Highway Commission for an underpass that will take the new Bay Shore highway, now being constructed on the east side of the railroad, under the tracks of the Southern Pacific R.R. at South San Francisco. The underpass will eliminate two bad grade crossings in the present bay shore route. If the railroad, the city of San Francisco and the State Highway Commission agree as to the apportionment of cost this structure is to be included in the construction budget of the highway commission for 1926.

The underpass is to have a clear roadway width of 40 ft. and at one side a separate subway 10 ft. above the grade of the road to provide for an 8-ft. sidewalk. Space will be provided over the underpass for 16 railroad lines although only eight tracks are to be laid at the time of construction. The alignment of the highway approaching the underpass will be in a 1,000-ft. radius curve on the San Francisco end and on tangent at the opposite end. Very easy grades will be used.

The roadway in the underpass at the lowest point will be 10 ft. below high tide level. Here the roadway will be carried on a thick slab of concrete extending the full width of the subway and decreasing in thickness as it extends up the grade of the approaches. The clearance in the closed portion of the underpass will be 14 ft.; a distance of 5 ft. is contemplated between clearance level and base of rail. The total length of the underpass and graded approaches will be 1200 ft.



SKETCH OF PROPOSED UNDERPASS AT SOUTH SAN FRANCISCO

Working Stresses Discussed at Two Meetings

(*Concluded from p. 813*)

they are likely to be revised upward. The great advances in the field control of concrete quality, said Mr. Lindau, promise to lead to the point where concrete will be the most uniform of structural materials.

Dr. D. B. Steinman, in an exhaustive review of the factors affecting steel working stresses, strongly endorsed the principle of leaving in the safety margin only those uncertainties of stress that it is not possible to eliminate. Seconding the reasoning presented by the A.S.C.E. committee, he concluded that upward revision of steel stresses is warranted. The bending stress of 20,000 recommended by that committee he described as conservative in view of the nature of beam action and the recommended reduction for unbraced length, while the base stress for columns and the reduction formula are consistent and safe. As the least figure acceptable for modern conditions, he would set the "amply conservative" 18,000-lb. figure.

Stresses in Timber

J. A. Newlin, of the Forest Products Laboratory, reviewed and defended in great detail the recommendations of the Department of Commerce committee for timber stresses, and outlined a process of calculation from test data by which the working stresses were fixed. He showed by graphic exhibits the principal elements of variation in timber strength, and the nature and amount of this variation as summarized from many thousands of tests.

E. F. Kenney laid strong emphasis on the uniformity of structural steel quality, as warranting liberal working stresses, but cautioned against putting faith in the yield-point determination. J. Loewenstein pointed out the dissimilar position of wood, concrete and steel as regards weight efficiency, and suggested that this is of more importance in determining relative fields of use than working stresses. G. L. Taylor referred to the fact that the Building Code committee of the Department of Commerce adopted practically in full the joint committee report as its rule for concrete working stresses, but did not adopt the steel recommendations of the American Institute of Steel Construction; it should in fairness accept the latter as the best judgment of the industry. L. D. Rights discussed particularly the column formula recommendations of the A.S.C.E. committee and stated his belief that they are better than the recommendations of the column committee seven years ago, because of advance in knowledge. He also pointed out how the recommendations of the A.S.C.E. committee are protected by provisos and in consequence are conservative. C. E. Chase dwelt at some length on variations of opinion in the A.S.C.E. committee, which gave rise to a divided report, and deplored the absence of better information on yield point of structural steel. A warning note was sounded by L. S. Moisseiff: In the general era of good feeling prevailing at present, are we not possibly inflating working stresses as we are inflating money values? He referred back to the failure of the first Quebec bridge, and traced it to a tendency toward high stresses something like that existing at present. Careful design is necessary in any event, he declared, but we should not build up an engineering Utopia as a setting for higher working stresses.

Prof. Hardy Cross, basing on the principle that design practice and specifications necessarily have close relation to working stresses, entered into sharp criticism of a number of specific details of the joint committee's design specifications for concrete.

Philadelphia Meeting

The Philadelphia meeting was opened by C. E. Chase with a review of the existing situation. F. C. Cartwright, secretary of the Department of Commerce Building Code Committee, then sketched the method of procedure of this committee and indicated the chief results of the revision of the first draft of that committee's working stress report. A new formula for timber columns, of fourth-power parabola type, has been introduced. Comments received from engineers, not fabricators, represented a vote of 2 to 1 in favor of 18,000 for basic steel stress, and accordingly this figure is considered approved. The steel column formula has also been raised.

F. E. Schmitt, of *Engineering News-Record*, pointed out the many complexities and uncertainties in strength determination and service estimates, that affect the design of the structure and therefore indirectly the working stresses. He then described the main considerations that controlled the A.S.C.E. working stress committee in recommending a 20,000-lb. stress, and called attention to the rather common but fallacious belief that working stresses can be deduced by some mathematical process. The A.S.C.E. committee report relates wholly to structural steel of present-day grade, however, and gives no key to the manner of selecting working stresses for higher strength steel, which may come into use in the future.

M. E. Hibbs, building inspector of Philadelphia, stated that the vast majority of buildings are not designed by engineers and referred to various conditions that steadily cut down the reserve of strength in buildings, among them the serious reduction of brickwork requirements, so that nowadays walls are almost eliminated and only piers are left, and these in turn are based on dimensions set by concrete practice. He made the claim that "all engineering should be done by engineers," although he has observed that even reputable engineers at times "forget" some of the dead-load. As to the building code committee recommendations, he challenged the recommendation of two separate column formulas, and called for caution in timber stresses as wood is not getting any better. S. C. Hollister, supporting in general the recommendations of the Joint Committee on Concrete and Reinforced Concrete, held that we must have competent engineering or specifications are not feasible. The joint committee recommendations have this basis; they are written for good practice, not for mediocre practice.

Commerce Commission Stresses

J. R. Worcester, of the Department of Commerce committee, spoke in support of the steel stresses recommended by that committee. On concrete stresses, he said that the committee has been criticised for adopting the joint committee report, but he believes that this course is right. W. A. Slater also discussed concrete stresses, and in particular stated that after a careful study of the data of 29 failures of concrete buildings, he found that none was attributable to high working stress. F. M. Masters raised question as to the quality of steel. This material has not improved with the advance of time, he said, except in the claims of its advocates, while concrete has been steadily improved through development of methods of preparation. He urged that a stronger steel be introduced.

Richard L. Humphrey praised the joint committee report as based on many supporting data and implied question as to whether steel stresses have been studied on an equally satisfactory basis. He emphasized that commercial controversy is responsible for the present working stress agitation.

In further comment, A. B. Richardson criticised the web-shear rules of the American Institute of Steel Construction as dangerous, especially where beams rest on walls and are not built in. W. Fisher declared it to be the height of folly to increase unit stresses, and said that steel stresses are already too high. H. H. Quimby expressed the unconventional belief that dead-load is more severe on structures than live-load, and charged that the Quebec bridge failure was the result of an increase of unit stresses, which should be a caution at the present time. T. S. Mercer corrected this statement as to the Quebec bridge failure by reference to the facts.

In answer to question raised by one of the members as to why the matter of incompetence, instead of being dealt with by low working stresses, should not be corrected by eliminating the incompetent engineer, Mr. Hibbs expressed the opinion that the incompetent man must live, and that accordingly construction rules must take him into account.

Longer Rails Approved for American Railways

The American Railway Association has announced that new specifications increasing the length of standard rails from 33 to 39 ft. have been approved by that organization. This change will result in a 16 per cent reduction in the number of rail joints and a saving of about one-sixth of the total of bolts, nuts, angle bars and spring washers now required.

Engineering Societies

Calendar

Annual Meetings

CITY MANAGERS ASSOCIATION, Lawrence, Kansas; Annual Convention, Grand Rapids, Mich., Nov. 17-19, 1925.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, Washington, D. C.; Annual Convention, Detroit, Mich., Nov. 18-20, 1925.

HIGHWAY RESEARCH BOARD, NATIONAL RESEARCH COUNCIL, Washington, D. C., Annual Meetirg of the Highway Research Board, Washington, D. C., Dec. 3-4, 1925.

AMERICAN ROAD BUILDERS ASSOCIATION, New York City; Annual Convention, Chicago, Ill., Jan. 11-15, 1926.

ASSOCIATED GENERAL CONTRACTORS OF AMERICA, Washington, D. C.; Annual Meeting, Portland, Ore., Jan. 18-23, 1926.

AMERICAN SOCIETY OF CIVIL ENGINEERS, New York City; Annual Meeting, New York City, Jan. 20-22, 1926.

ENGINEERING INSTITUTE OF CANADA, Montreal; Annual Meeting, Toronto, Ont., Jan. 27-29, 1926.

AMERICAN CONCRETE INSTITUTE, Detroit, Mich.; Annual Meeting, Chicago, Ill., Feb. 23-26, 1926.

The Iowa Section of the American Water Works Association will hold its annual meeting at Dubuque, Iowa, Dec. 2-4 at the Julien Dubuque Hotel. J. W. McEvoy, superintendent of water-works, Dubuque, is chairman of the local committee.

The Providence Section of the American Society of Civil Engineers and the Municipal Section of the Providence Engineering Society held a joint meeting Nov. 9, the address on Metropolitan Traffic Problems being given by Robert H. Whitten, expert on zoning.

The First Missouri Conference on Water Purification was held at Kirksville, Mo., Oct. 26 and 27 with various cities represented. A. V. Graf, chemical engineer, St. Louis Water Department, was elected as president and G. W. Putnam, chief engineer of the State Department of Health, Jefferson City, Mo., as executive secretary.

Personal Notes

C. F. JOHNSON, who was recently put in charge of the Tarpon Springs, Fla., branch of the Hunter Engineering Co., as stated in these columns Oct. 22, p. 693, writes that his name was incorrectly given there as E. F. Johnson.

LOUIS F. WARRICK, of Charleston, W. Va., has been appointed assistant state sanitary engineer in the Wisconsin engineering department at Madison. Mr. Warrick is a graduate of West Virginia University in chemical engineering and took three years' postgraduate work at Cornell University in sanitary engineering.

PROF. W. K. HATT, professor of civil engineering at Purdue University and recently for two years director of the work of the Advisory Board of Highway Research at Washington, D. C., has been appointed by the Board of Trustees of the Indiana World War Memorial as consulting engineer to the board.

THE PIKE CONSTRUCTION CO., street and road contractor, at Wabash, Ind., has changed its name to the Mid-West Asphalt Paving Co. No change other than the name is now made in the company. Last April D. A. Pike retired from the organization to transfer his construction activities to Sarasota, Fla.

HOWELL J. DAVIS has been appointed city manager of Knoxville, Tenn., to serve during the absence of Louis Brownlow who is on sick leave.

W. G. BLACK, who for the past two and a half years has been state engineer and chief engineer of the North Dakota State Highway Commission, has resigned and will act as manufacturers' agent for a limited line of municipal and highway materials, with headquarters at Bismarck, N. D.

H. C. FRAHM, formerly division engineer for the North Dakota State Highway Commission in the Minot, N. D. district, has been appointed state engineer and chief engineer of the commission, succeeding W. G. Black, resigned.

J. W. SWAREN, recently engineer in the Treasury Department at Washington, D. C., has opened an office there for consulting service to other engineers who may be engaged in the preparation of data for submission to federal departments and bureaus. Prior to the war, Mr. Swaren was engineer of tests with the Pacific Gas & Electric Co. and from 1908 to 1917 was engaged in private practice in San Francisco. Subsequent to the war, he was for a time general sales manager and engineer of the Duriron Co. of Dayton, Ohio, before going to the Treasury Department. More recently he has been in private practice in appraisal and valuation work.

ROBERT B. VAN HORN, maintenance engineer, Tieton Division, Yakima Project, U. S. Bureau of Reclamation, has resigned to accept a position as instructor in the engineering department of the University of Washington. His place has been filled by transfer of TOM C. MEAD from the storage division.

C. R. WEYMOUTH, formerly division engineer of Division 2, Milwaukee, Wis., has been appointed design engineer of the Wisconsin Highway Commission, filling the vacancy caused by the recent resignation of H. J. Kuelling. Mr. Weymouth studied engineering at the University of Wisconsin and the University of Maine, and before entering work with the Wisconsin Highway Commission in 1915 was connected with the U. S. Reclamation Service on hydroelectric development projects in the West. To replace Mr. Weymouth in Division 2, H. W. VROMAN is transferred from the position of division engineer of Division 6, Eau Claire, Wis.; and W. F. BAUMGARTNER, assistant, becomes division engineer in Division 6.

Obituary

FRANK B. MCCORD, Flushing, Long Island, N. Y., for thirty years connected with Post & McCord, steel contractors, New York City, and for a long time treasurer of the company, died Nov. 6 in Morristown, N. J., aged 48 years. The firm was established by Mr. McCord's father, the late William H. McCord, and the late Andrew J. Post, grandfather of the present president, Andrew J. Post.

THOMAS H. TRACEY, former city engineer of Vancouver, B. C., and of London, Ont., died at Vancouver Oct. 31 in the 77th year of his age. At the time of his death he was a member of the municipal council of Vancouver. Previous to the time that he served as city engineer of London, Ont., some thirty years ago, Col. Tracey was associated with the late George Durand, architect, upon the building of the state capitol at Albany, N. Y. During the Northwest rebellion in Canada in 1885 he commanded a company of the 7th Battalion.

FREDERICK B. PECK, professor of geology and mineralogy at Lafayette College, Easton, Pa., died Nov. 2. Professor Peck was a graduate of Amherst College in the class of 1886 and has been at Lafayette since 1901.

HENRY C. HODGKINS, consulting engineer, Syracuse, N. Y., and for a long period member of the firm of Moffett, Hodgkins & Clarke Co., builders of seventy-five water-works and a number of central lighting stations and gas works as well as two steam railroads, died Nov. 7 at the age of 71 years. Mr. Hodgkins was a graduate of Union College in 1875. His early work was in connection with St. Lawrence River parks. From 1883 to 1885 he was city engineer of Watertown, N. Y. Besides his connection with Moffett, Hodgkins & Clarke Co., he was consulting engineer on many water power and municipal water supply projects. He was the author of many papers read before the American Water Works Association.

MICHAEL J. FEENEY, Pittsburgh, Pa., general superintendent of Booth & Flinn, Ltd., contractors, New York and Pittsburgh, and for forty years associated with the firm, died Nov. 6 in the Penn hospital, Pittsburgh, at the age of 73. Mr. Feeney was born in England in 1852 and came to the United States with his parents when he was four years old. His first work was with the Standard Oil Co., at Pittsburgh, followed by his long connection with Booth & Flinn.

From the Manufacturer's Point of View

A Point of Contact
Between Maker and User of
Construction Equipment and Materials

International Trade Exhibit Opens in New Orleans

Complete Marketing Service to Be Provided for Manufacturers of All Countries

THE International Trade Exhibition, the permanent exhibition of materials and equipment from the several states and foreign countries, is now open in New Orleans and exhibits are placed in one of the three 500,000 sq.ft. building units furnished it by the United States Government. A proclamation of President Coolidge, dated March 13, 1925, has been sent to all the states and all foreign countries asking them to participate in the exhibition. The International Trade Exhibition is a non-profit domestic corporation formed for the purpose of fostering better trade relations between the peoples of the United States and of foreign nations. In addition to opening foreign sales outlets, the promoters of the exhibition believe that it provides a means for a more intensive development of the New Orleans trading territory. "The exhibition offers a unique, well-rounded marketing service where buyers and sellers of raw and fabricated products of all countries can transact business to a mutually satisfactory conclusion."

SERVICE FEATURES

An outline of the service angles of this project is as follows: 1. Exhibitors may maintain permanent offices. 2. Exhibitors may maintain exhibits with or without their own representatives in charge. The exhibition management will undertake to arrange representatives on a whole or part time basis. (Although goods from foreign countries come into the United States duty free as exhibits, duties must of course be paid if and when the commodities comprising these exhibits are sold in the United States.) 3. There will be a retail display and sales floor where manufacturers who are not entrenched with retailers serving the New Orleans territory may do introductory selling to the consumer. Economical warehouse facilities are available nearby for reserve stock. 4. A reservation of space is provided where the inventors of the world may exhibit their inventions. 5. A general commercial reference library to enable prompt response for those interested in securing direct connections is also part of the plan. A trade opportunities bureau is an important work of this division. 6. Other services are being considered, such as letters of introduction from headquarters to visitors desirous of contact with firms in other centers, either direct or through the proposed regional offices.

It is the desire of the managers of the exhibition to so firmly establish the general mark "Intrex" that the use of this mark by an exhibitor on his stationary, commodities, etc., will at once create or sustain confidence in the proposition or commodity offered.

In order to further the interests of all exhibitors and intercountry trading in general, an international advisory council is in the process of formation. In addition to representation on the council, the United States will have three regional councilors, Eastern, Mississippi Valley Basin, and Western. State advisory boards are also being formed.

H. Walton Heegstra, the director-general, states that this institution does not supplant any work or service of any existing trade development, organization or bureau. "In the spirit of service, it is interdependent with them, not independent."

Equipment and Materials

Back-Filler Attachment Is Four Tools in One

The back-filler attachment manufactured by the Killefer Manufacturing Co., Los Angeles, Calif., and designed to fit Caterpillar tractors, is adaptable to use not only as a back-filler, but also as a bulldozer, a grader, and a snow plow. When used as a back-filler, it will fill trenches on the right or left, the change from one side to the other being made entirely by turning the tractor. The driver does not have to leave his seat. By locking the blade straight across the top, this attachment becomes a bulldozer capable of smoothing and leveling truck dumps or fills and similar rough land. Its uses as a grader are claimed to be as follows: For subgrade finishing, since it will oscillate freely on grade guides for

foundation of concrete, gravel or other roads; as a gravel or material distributor; as a blader for shouldering hard surfaced roads; and as a general dirt or gravel road maintainer. When fitted with a special blade, it may be used as a snow plow.

It is claimed that this attachment increases the traction of the power unit rather than cutting it down, for the weight is on the tractor and not on the ground. The desired amount of ground pressure on the blade is adjusted by means of weights on levers. When the tractor is turned around, the blade can be automatically shifted to the opposite side, if desired. Side draft is equalized since both driving members of the tractor pull equally and steer normally. It is not necessary to push this blade with one track to offset side draft.

Heavy Duty Material Stacker Is Fast in Operation

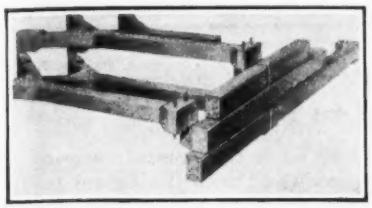
Designed especially for use in construction warehouses and similar places, the type PB electric power stacker manufactured by Lewis-Shepard Co., Boston, combines a rugged construction with fast operating mechanism. The machine operates with a 2½-hp. motor from a power line and develops a lifting speed of 40 ft. per min. with a 1,000-lb. load. It is built in 1,000 and 2,000 lb. capacities.

This stacker is claimed to be the first and only portable elevator using a worm drive. The worm runs in a bath of oil and there are no exposed gears; operation both up and down is effected through a controller. Other advantages claimed for this stacker are safety and ease of operation. Safety is taken care of by providing a brake that is always on except when released by the operator and by safety-hinge locks that prevent the top section from falling back. Ease of operation is provided by Hyatt roller bearings in the floor wheels and sheave wheel and by a free steering handle that operates in a very small amount of space.

Cribbing Unit Forms a Stable Interlocked Retaining Wall

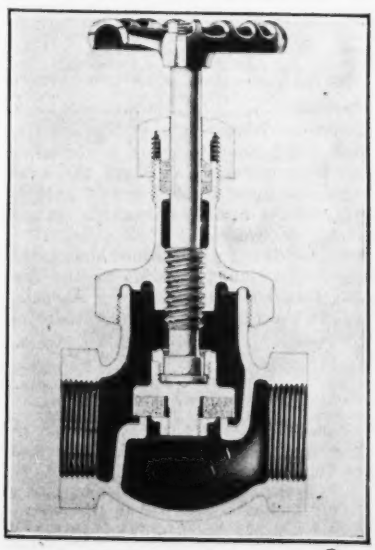
The new concrete cribbing units manufactured by the Federal Cement Tile Co., Chicago, in addition to performing their intended purpose as retaining walls provide a finished wall on the face. The unit consists of two parts, a face member or stretcher and an anchor member or header; the latter is attached to the rear of the stretcher by lugs. These lugs project 6 in. to the rear from each end of the stretchers which are 6 ft. long, 1 ft. high and 4½ in. thick. The headers are Y-shaped members having a stem and legs 8 in. by 4 in. in section and an enlarged base or anchor detail fitted with two ¾ in. round bars which project four inches from the top and bottom; these bars fit into holes provided in the corresponding anchor lugs of the adjacent stretchers.

In laying the wall the joints in alternate rows of stretchers are staggered while the headers are placed with their bases resting on top of the anchor lugs at each joint. Thus the header is connected to the adjacent stretchers of alternate courses while the end of each header rests behind the middle of the stretcher in the intermediate course. Furthermore, the spread legs at the rear of the headers provide the anchorage for the structure as a whole and are arranged so that the ends of the legs of one course rest on the ends of the legs of the course next below. Through this arrangement the headers are afforded the necessary support at the rear and the introduction of a rear row of stretchers as required in some other forms of cribbing construction is unnecessary.



Medium Pressure Valves Have New Features

The new line of medium pressure bronze valves in both globe and angle designs, manufactured by Jenkins Bros., New York City, has been designed to fulfill a need for a valve with a renewable disk which will satisfactorily meet higher pressures than recommended for standard pattern valves. A feature of these valves is the bonnet and union made in one piece to screw onto the outside of the body threads. This construction gives added strength to the body end and the bonnet hexagons, being made specially large, allow easy removal of the bonnet without distortion. Another new feature is the "ball joint" formed when the bonnet and body are screwed together. This is claimed to make a very strong, though easily removed joint.



These valves are regularly fitted with composition disks for high pressure work. It is claimed that these are superior to a metal-to-metal seat for this purpose. No regrinding is necessary to insure a tight valve. The spindle is made of manganese bronze, and the stuffing box is deep, containing a large amount of asbestos packing which is compressed by means of a bronze follower. A ventilated hand wheel of malleable iron is used. The valve is furnished with screwed or flanged ends in sizes ¼ in. to 3 in.

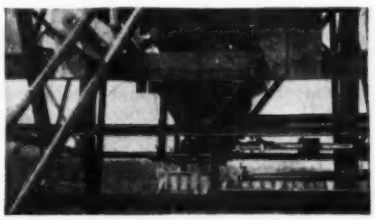
New Roofing Unit Is Light, Fireproof and Durable

A roofing unit consisting of a steel supporting member covered with any standard pressure resisting insulating material over which is applied a built-up bituminous waterproofing such as asbestos or rag felt covered with gravel, is being manufactured by Holorib, Inc., Cleveland. The steel supporting unit as shown in the illustration forms a light weight roof deck strengthened by the closed triangular ribs. It is built in various lengths to form a continuous roof surface. Standard Holorib sheets are furnished in 22, 24 and 26 gage copper bearing steel, painted and baked. Sheets are also rolled of lead coated steel or galvanized metal. The roofing can be safely applied over purlins spaced at a maximum of 4 ft. c. to c. Special clips secure the roof deck to the purlins. The roofs allow for ample expansion and contraction so that the danger of cracked waterproofing or insulation is minimized.

Equipment Measures Concrete Aggregates by Weight

As a result of the tendency in certain localities to specify that concrete aggregates shall be measured by weight, the Blaw-Knox Co., Pittsburgh, Pa., has patented a measuring apparatus consisting of a weighing hopper. This hopper is suspended beneath the sand and stone openings of the bin. It is so arranged that both the fine and coarse aggregates are discharged into it, being separated from each other by means of a steel partition. The weighing hopper is connected to a standard beam scale by an overhead system of levers, the whole arranged for one-man operation.

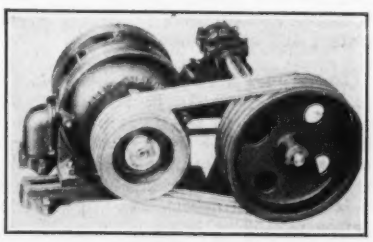
In using the apparatus, the scale is set for the correct weight of the stone. After this material has been weighed, the scale is set for the combined weight of both sand and stone, and sand admitted until the scale balances. The materials are then discharged into the truck or receiving hopper of the mixer through a latched counterweight bottom-dump door.



New Short-Center Drive Designed to Prevent Belt Slip

The new short-center drive for power transmission manufactured by the Allis-Chalmers Manufacturing Co., Milwaukee, Wis., consists of two grooved sheaves and a number of specially constructed "V" belts. The sheaves are set just far enough apart so that the belt fits the grooves without either tension or slack. These belts are known as Texrope, and since they just fit the sheaves, there is claimed to be no slack or lost motion in the drive. Also, the "V" construction is claimed to eliminate slip, since the harder the pull the more firmly the belts grip the grooves. Since they are elastic and stretchable, they do not jerk either in starting or running, nor do they transmit vibrations, but act as cushions to the driving and driven machines.

Bearing pressures are low since no belt tension is employed. The drive occupies very little space, is silent,



clean, unaffected by moisture or dirt, safe, simple and trouble-proof. Since there is no slip, the speed ratios are fixed and exact. Texrope drives from ½ to 250 hp., with ratios up to 7 to 1, and belt speeds from 800 to 6,000 ft.p.m. have already been placed in service.

Publications from the Construction Industry

Conveying Machinery—LINK BELT Co., Chicago, is celebrating its 50th anniversary this year and has contributed an interesting and instructive booklet, "The Story of Link Belt, 1875-1925," to engineering-industrial literature. The booklet tells the story of the detachable link chain from its inception by W. D. Ewart in 1874, through its first offering to the public provided by the incorporation of the Ewart Mfg. Co., the forerunner of the present Link-Belt Co., to its present use in elevating and conveying equipment of all kinds as manufactured in the ten plants of the Link-Belt Co. situated throughout the United States.

Diamond Drills — SULLIVAN MACHINERY Co., Chicago, has issued its catalog No. 80, containing 80 pp. descriptive of the Sullivan line of diamond drills. The book includes specimen lists of diamond-drill equipment, instructions for selecting an outfit, and a chapter on setting up and operating the machines under ordinary field conditions. There is also a section on setting diamond bits.

Vitrified Clay Sewer Pipe — CLAY PRODUCTS ASSOCIATION, Chicago, has issued a 24-p. booklet entitled "Tentative Standards," the purpose of which is to secure the co-operation of engineers, architects and others interested in drainage and building materials in the general effort for standardization and elimination of varieties of special designs in clay sewer pipe. All manner of clay sewer pipe and fittings are illustrated with all standard sizes and detail dimensions listed in tabular form below. Several pages in the back of the booklet are devoted to specifications for the laying of vitrified clay sewer pipe. These specifications cover excavation, bracing and shoring, foundations, pipe laying, joints and backfilling.

Electric Rivet Heaters — AMERICAN HOIST & DERRICK Co., St. Paul, Minn., features its line of electric rivet heaters and shop appliances in a new booklet just issued. Details of design and construction are given, together with performance comparisons of electric rivet heaters and oil furnaces.

Centrifugal Pumps — LECOURTENAY Co., Newark, N. J., in bulletin 2040, devotes 15 pp. to the description of its line of single-stage pumps. Each separate model is illustrated and briefly described. Several pages are devoted to a detailed description of double suction pumps.

Rock Working Machinery—STURTEVANT MILL Co., Boston, Mass., in its new catalog describes such construction equipment as jaw rock crushers, bucket elevators, screens both rotary and vibrating, and belt conveyors. Many illustrations add to the usefulness of the catalog.

Business Side of Construction

FACTS AND EVENTS THAT AFFECT COST AND VOLUME

This Week's Contracts—Week Ago—Same Week Last Year

THE money value of contracts reported in the present issue of *Engineering News-Record* is here compared with the figures for corresponding weeks. Minimum costs observed are: $15,000 for water-works and excavations; $25,000 for other public works; $40,000 for industrial and $150,000 for commercial buildings.

Money Value of Contracts Let—Entire U. S.

Week Ending	Public Work	Private Work	Total Contracts
November 12, 1925	$13,866,000	$35,810,000	$49,676,000
November 5, 1925	15,962,000	29,929,000	45,891,000
November 13, 1924	22,248,000	13,091,000	35,339,000
Heaviest Week			
1925, Sept. 3	16,215,000	69,424,000	85,639,000
1924, May 8	26,370,000	38,829,000	65,199,000
January 1 to Date			
1925	936,195,000	1,293,085,000	2,229,280,000
1924	890,750,000	912,659,000	1,803,409,000

Production of Basic Materials at Higher Rate Than a Year Ago and on the Up Trend in Most Cases

Increase Should Exert Restraining Influence Upon Rising Prices—October Steel Output Up—Gain in Unfilled Tonnage

THE output of basic materials, thus far in 1925, has shown very much the same general characteristics as that for the year preceding, except that current production is in greater volume.

The present rate of mill operations is above a year ago and slightly under that for the corresponding period in 1923.

Last year showed a gradual slowing down during the first half, with a steady gain from mid-summer on into the first quarter of 1925.

This year showed the same decline during the first half, with a brisk reaction at the beginning of the second, which may prevail throughout the remainder of the year despite the suspension of production at anthracite coal mines.

Aside from the seasonal decline in cement and brick production and a slight drop in lumber cutting, reports show increased activity in iron, steel, bituminous coal, textiles and automobiles.

The production situation in four of the principal basic building materials is as follows:

Steel—As calculated by the American Iron and Steel Institute, the output for the nation's mills was 3,892,946 tons in October, against 3,492,904 in September and 3,125,418 tons for October, 1924. This year (Jan. 1 to Nov. 1) is 20 per cent ahead of last in production of steel ingots.

Unfilled orders on books of the U. S. Steel Corporation, Oct. 31, totaled 4,109,183 tons, against 3,717,297, Sept. 30 and 3,525,270 tons on Oct. 31, 1924.

INGOT OUTPUT—ALL COMPANIES

	1925 Tons	1924 Tons
January	4,198,564	3,649,913
February	3,756,243	3,826,246
March	4,198,520	4,206,699
April	3,587,524	3,348,466
May	3,458,253	2,640,034
June	3,207,056	2,065,676
July	3,087,590	1,877,789
August	3,424,034	2,552,891
September	3,492,904	2,827,625
October	3,892,946	3,125,418
10 months	36,303,634	30,120,757

Lumber—While actual production shows a tendency to decline, according to latest reports which cover the week of Oct. 31, there is evidence of gain in new business and in shipments on old orders.

The following tabulation shows the lumber situation, according to the National Lumber Manufacturers Association, for the three weeks indicated:

	Week of Oct. 31 Ft. b.m.	Corresponding Week in 1924 Ft. b.m.	Preceding Week in 1925 Ft. b.m.
Production	225,884,128	210,294,042	240,047,481
Shipments	232,155,319	227,897,215	228,907,238
New business	215,114,779	222,809,603	213,610,339

The lumber movement for the first forty-four weeks of 1925, follows:

	Production Ft. b.m.	Shipments Ft. b.m.	Orders Ft. b.m.
1925	10,6 4,008,664	10,559,424,262	10,340,022,443
1924	10,215,088,869	10,157,742,628	9,841,531,523

(*Continued on p.* 821)

Mass Curves of Contract Values for Years 1922-1925 and a Projection of the Curves to 1930

ON THE generally conceded assumption that the construction of public works has not kept pace with public needs, the projection of the non-buildings curve is a continuation of the 1925 rate of contract-letting. The projection of the buildings curves are at the average rate of lettings in the years 1921-1925. This is regarded as safely conservative, as the period includes 1921, the depression year. This average rate is taken, instead of the boom 1925 rate, because it is unlikely that the present rate will be maintained.

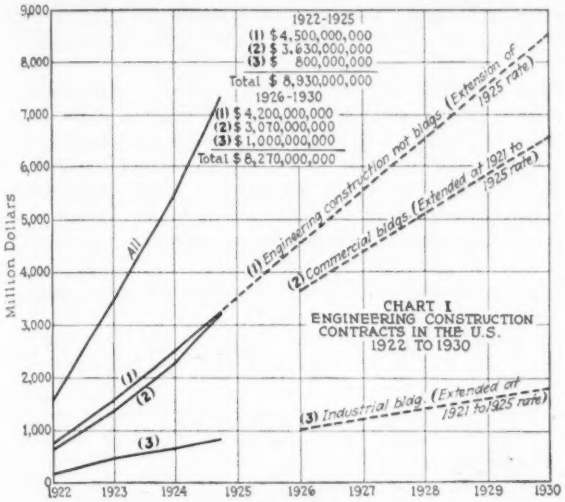
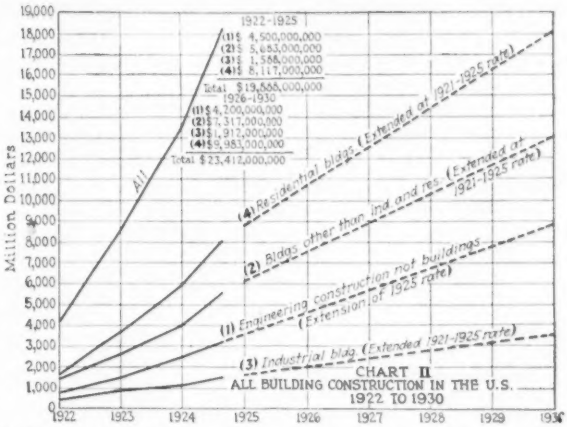


CHART I

Engineering construction contracts of the following minimum costs: Water-works, excavation, etc., river improvement, $15,000; other public works, $25,000; industrial buildings, $40,000; commercial buildings, $150,000. The total five year estimate (1926-1930) is $8,270,000,000, of which about half is for buildings.



Note: It is of first importance to understand that these are mass curves and that the 1930 reading is the total for the period 1922 to 1930

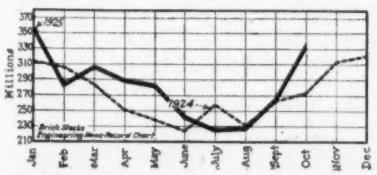
CHART II

All building construction in the United States and the non-buildings curve of Chart I. The buildings projections are at the average rate in 1921-1925. The aggregate in the next five years is $23,412,000,000, of which 40 per cent is residential. The annual rate for buildings only is $3,840,000,000.

Production of Basic Materials at Higher Rate Than a Year Ago

(Continued from p. 820)

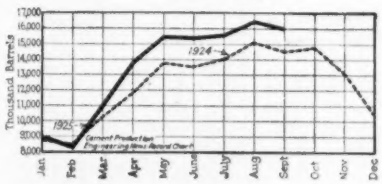
Brick—Unfilled orders at yards show a slight drop, according to latest reports of the Common Brick Manufacturers Association. Shipments on old orders and reserve stocks at yards throughout the country, are substantially greater. Compared with this

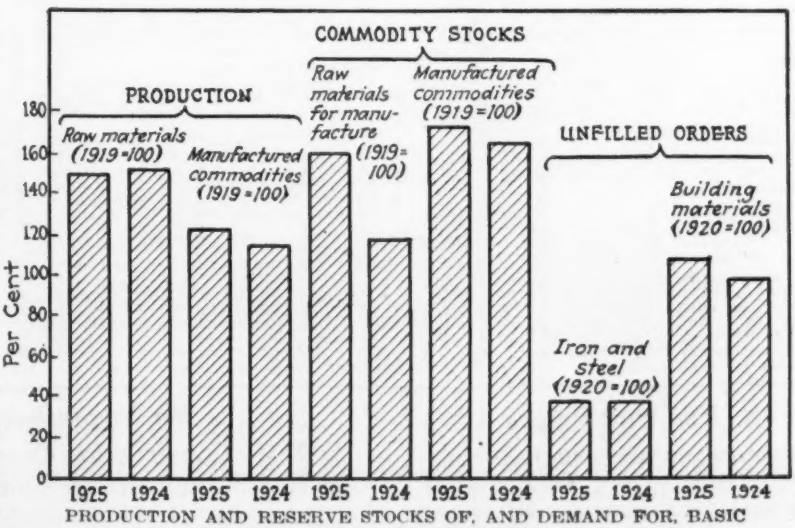


time last year, there is an all-around gain in the common brick movement despite the seriousness of the fuel shortage. The following tabulation shows the situation as of Oct. 1, 1925:

1924	Burned Brick on Hand M	Unburned Brick on Hand M	Brick Moved from Yards During Month M	Orders on Books M
Oct. 1	272,172	61,150	111,846	226,529
1925				
Jan. 1	354,477	51,162	118,759	202,906
Feb. 1	279,862	23,951	94,185	201,479
Mar. 1	305,831	34,891	110,790	247,176
Apr. 1	287,800	43,446	170,697	329,673
May 1	281,858	71,266	206,551	339,629
June 1	239,389	67,480	180,851	333,967
July 1	225,401	82,987	173,215	326,226
Aug. 1	227,306	92,267	180,407	292,775
Sept. 1	265,897	89,608	159,309	301,913
Oct. 1	338,857	104,066	171,830	252,511

Cement—Production and mill shipments are now on the regular seasonal down-trend which usually results in a reaction during the month of February. Mill reserves are consequently on the increase and will probably continue so until March. The chart to the right is according to the Bureau of Mines.





PRODUCTION AND RESERVE STOCKS OF, AND DEMAND FOR, BASIC MATERIALS DURING MONTH OF SEPTEMBER—U. S. DEPT. OF COMMERCE FIGURES, E. N.-R. CHART

Weekly Construction Market

COST of construction, as shown by the *E. N.-R.* Index Number, is less than one per cent higher than it was at this time last month.

There is one factor working toward the continuation of the present trend and two exerting pressure in the opposite direction.

Lumber is the main item in the current market, tending to advance the cost of building. The extremely heavy demand for Southern pine in the Florida boom area is causing a scarcity of this material in other sections of the country.

Rate of production and size of reserve stocks in most of the basic materials are sufficiently great to prevent an abrupt advance in prices, despite a demand unprecedented for this season of the year.

The other important move tending toward lower construction cost is a drop of 10c. per bbl. in portland cement at four midwestern mills. This follows a similar decline in eleven other points in the same territory during the past month.

The labor situation in each of the twenty-one cities reporting regularly to *E. N.-R.* is tranquil, with the following five exceptions: Dallas carpenters are preparing to demand a wage increase of $1 per day; New York bricklayers are asking $18 per day with a five-day week, there is also a factional disturbance arising among union ironworkers; Philadelphia anticipates wage demands on the part of some of the building crafts; Seattle and Puget Sound district reports slackening of demand for building trades mechanics, also that demand for men at lumber camps and mills will absorb much of the surplus; Boston ironworkers returned to work at the old rate of $1.10 per hr. after having gone on strike for an advance of 15c. per hr.

	New York	Atlanta	Dallas	Chicago	Minneapolis	Denver	San Francisco	Seattle	Montreal
Steel Products									
Structural shapes, 100 lb.	$3.34	$3.90	+$4.15	$3.10	$3.35	$3.07½	$3.30	$3.35	$4.25
Structural rivets, 100 lb.	4.50	4.45	4.75	3.50	3.75	4.65	5.00	3.75	6.00
Reinforcing bars, ¾ in. up, 100 lb.	3.24	3.40	3.38	3.00	3.25	3.77½	3.35	3.25	2.30
Steel pipe, black, 3½ to 6 in. lap, discount	48%	55%	53.6%	51%	54.25%	36%	39.2@51%	50%	59.49
Cast-iron pipe, 6 in. and over, ton	50.60@51.60	49.50	54.00	49.20@50.20	53.00	63.00	53.00	55.00	55.00
Concreting Material									
Cement without bags, bbl.	2.50@2.60	2.25	2.05	2.10	—2.32	2.84	2.31	2.65	1.80
Gravel, ¾ in., cu.yd.	1.75	2.20	2.38	2.00@2.20	1.65	1.90	2.15	1.50	1.25
Sand, cu.yd.	1.00	1.65	2.00	2.00@2.20	1.25	1.00	1.50	1.50	1.25
Crushed stone, ¾ in., cu.yd.	1.85	2.20	2.83	2.00@2.20	1.75	2.50	2.15	3.00	1.90
Miscellaneous									
Pine, 3x12 to 12x12, 20 ft. and under, M.ft.	61.00	34.00	60.00	+54.50	35.75	33.75	27.00	25.00	60.00
Lime, finishing, hydrated, ton.	18.20	22.50	20.00	20.00	25.50	24.00	22.00	24.00	21.00
Lime, common, lump, per bbl.	2.25@2.75	1.30	1.85	1.30	1.35	2.70	1.60	2.80	9.50
Common brick, delivered, 1,000	17.50@18.50	10.50	11.10	12.00	13.50	12.00	15.50	15.00	17.50
Hollow building tile, 4x12x12, per block	Not used	.0893	.10	.075	.....	.085	.....	.10	.12
Hollow partition tile 4x12x12, per block	.1027	.0893	.10	.075	.07	.085	.108	.10	.10
Linseed oil, raw, 5 bbl. lots, gal.	+1.02	+1.12	1.13	1.01	1.11	1.23	—1.11	1.12	1.25
Common Labor									
Common labor, union, hour	.55@.75	.30	.....	.87½	.....	.50@.55	.55	.62½	.....
Common labor, non-union, hour	.....	.25	.30@.50	.82½	.45@.55	.35@.50	.50	.50	.25@.35

Explanation of Prices—Prices are to contractors in carload lots unless other quantities are specified. Increases or decreases from previous quotations are indicated by + or — signs. For steel pipe, the prevailing discount from list price is given; 45-5% means a discount of 45 and 5 per cent.

New York quotations delivered, except sand, gravel and crushed stone, alongside dock; common lump lime, in 280-lb. bbl. net and hydrated lime, f.o.b. cars; tile "on trucks"; linseed oil and cast-iron pipe f.o.b. Labor—Concrete laborers' rate, 93¾c.; building laborers, 75c.; excavating laborers, 55c. per hr.

Chicago quotes hydrated lime in 50-lb. bags; common lump lime per 180-lb. net. Lumber, sand, gravel and stone delivered on job; pine is 4-in. instead of 3-in.

Minneapolis quotes on fir instead of pine. Brick, sand and hollow tile delivered. Cement on cars. Gravel and crushed stone quoted at pit. We quote on brown lime per 180-lb. net; white is $1.70 for Kelly Island and $1.60 for Sheboygan. Common labor not organized.

THIS limited price list is published weekly for the purpose of giving current prices on the principal construction materials, and of noting important price changes on the less important materials. Moreover, only the chief cities are quoted.

Valuable suggestions on costs of work can be had by noting actual biddings as reported in our Construction News section.

The first issue of each month carries complete quotations for all construction materials and for the important cities. The last complete list will be found in the issue of November 5, the next on December 3.

Denver quotes on fir instead of pine. Cement "on tracks"; gravel and sand at pit; stone on cars; lime, brick, hollow tile and lumber on job. Tile price is at warehouse. Linseed oil, delivered in wooden bbl. Common lump lime per 180-lb. net.

Atlanta quotes sand, stone and gravel per ton instead of cu.yd. Common lump lime per 180-lb. net.

Dallas quotes lime per 180-lb. bbl. Steel, cement, cast-iron pipe and crushed stone f.o.b. cars, other materials delivered.

San Francisco quotes on Heath tile, size 5½ x 8 x 11½. Prices are all f.o.b. warehouses except C. I. pipe, which is mill price plus freight to railway depot at any terminal. Common lump lime per 180-lb. net. Lumber prices are to dealers in yards at San Francisco, for No. 1 fir, common.

Seattle quotes on Douglas fir (delivered) instead of pine. Lump finishing lime per 180-lb. net. Brick and hollow building tile delivered. Hydrated lime in paper sacks. Sand and gravel at bunkers.

Montreal quotes on pine lumber. Sand, stone, gravel and lump lime per ton. Stone and tile are delivered; sand, gravel, lime and cement on siding; brick f.o.b. plant; steel and pipe at warehouse. Hollow tile per ft. Cement price is in Canadian funds (the Canadian dollar stands at 100.062). Bag charge is 80c. per bbl. Discount of 10c. per bbl. for payment within 20 days from date of shipment. Steel pipe per 100 ft. net; 3½-in., $59.49.

For Unit Prices

covering trackwork on the Queensboro Subway, New York, and for sewers in Taylorville, Ill.,

see Construction News

On November 1, 1925

E. N.-R. Construction Cost Index Number 205.95

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For Explanation and Details of Indexes Since 1913 See the First Issue of Every Month